New England Water Works Association

ORGANIZED 1882.

Vol. XLVIII. March, 1934. No. 1.

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A TEN FOOT PARSHALL MEASURING FLUME AT PROVIDENCE.

BY CHAS. G. RICHARDSON.*

[*Read September 20, 1933.*]

While this paper relates to a particular sewage-metering problem the methods used in its solution are equally applicable for water under similar conditions and requirements.

For the last two years the Providence Sewage Disposal Plant at Fields Point has been undergoing gradual but major changes in anticipation

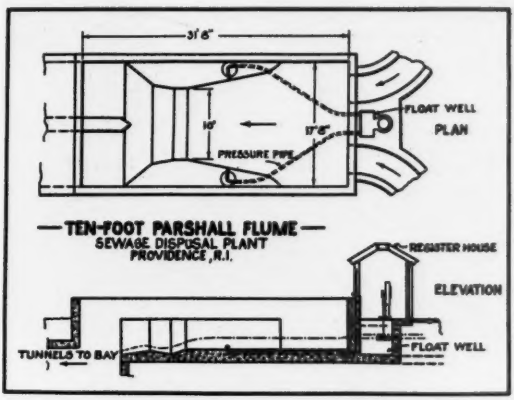

FIG. 1.— PLAN AND ELEVATION OF PARSHALL FLUME, PROVIDENCE, R. I.

of the day when it will emerge from chemical precipitation, begun in 1901, to the activated sludge process. Sewage from the entire Providence area arrives at the Ernest Street Pumping Station near the plant where it is lifted 27 ft. to the influent channel. After passing through various settling and clarifying tanks the effluent is discharged through large underground tunnels into Narragansett Bay 400 ft. distant.

* Builders Iron Foundry, Providence, R. I.

If the plant is to be operated efficiently and satisfactory treatment obtained, it is essential to measure the entire flow and record its hourly variations. At first it was proposed to install an ordinary weir in either the influent or effluent channel, but calculation revealed that in times of heavy storms or high tides the resulting backing up of the sewage might overflow

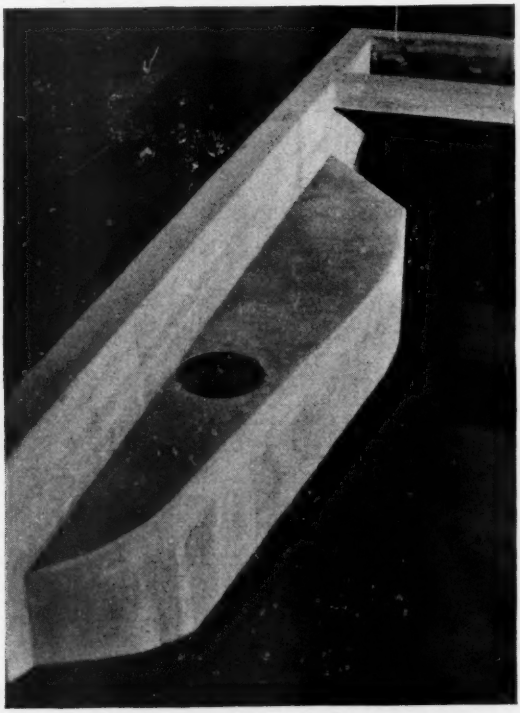

FIG. 2.— SIDE VIEW OF PARSHALL FLUME, PROVIDENCE, R. I.

screen chambers, manholes or process tanks. A large Venturi tube was considered but would have involved costly excavation due to the hydraulic gradient characteristics. Finally a Parshall Flume was suggested.

Since this device is little known in New England, brief mention of its development and characteristics may not be inappropriate. In 1915, V. M. Cone, curiously harmonizing his scientific investigations with his name, conducted experiments at the Colorado Agricultural College, Fort Collins, Col., upon an open, Venturi-shaped flume having a converging inlet, straight throat and diverging outlet, with level floor. In 1917 he published his results, which seemed to be promising. Other experimenters and hy-

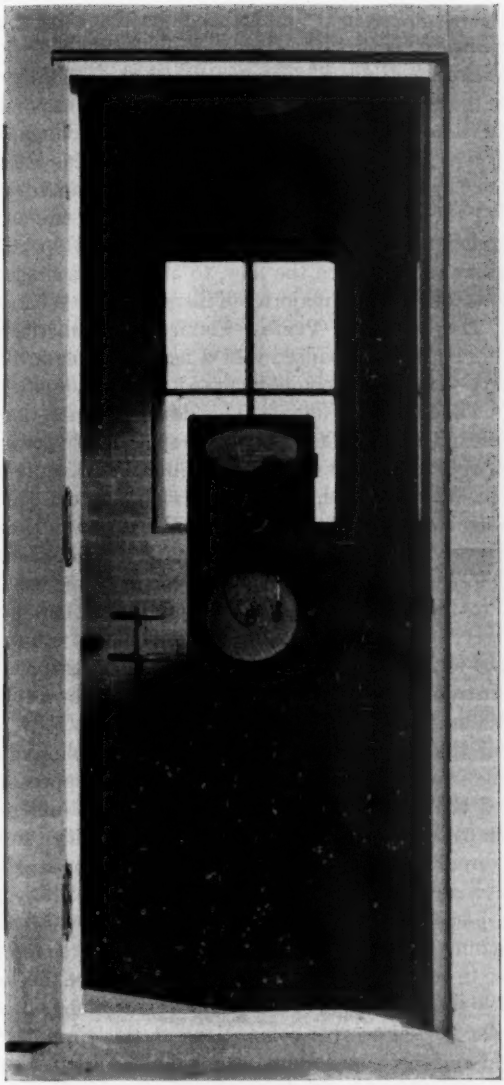

FIG. 3.— REGISTERING DEVICE FOR PARSHALL FLUME, PROVIDENCE, R. I.

draulic laboratories, notably the Cornell Laboratory, pursued the idea but gradually lost their enthusiasm. It seemed impossible to establish a definite, dependable relation between the difference in head at inlet and throat and various rates of flow through the flume. R. L. Parshall, Senior Irrigation Engineer, however, continued experimenting at the Colorado Station and after literally thousands of calibrations devised many important changes, including alterations in the angles of convergence and divergence and in the side slopes to give greater stability of the water surface. He also added a dip in the floor beyond the throat section to produce a hydraulic jump at this point. Of these changes the last was the most important since it reduces observing two gage heights (inlet and throat), which were previously necessary to determine the flow, to observing a single gage height in the inlet cone for the great majority of installations. While he called the new device "The Improved Venturi Flume," the American Society of Civil Engineers in 1930 announced that a new principle in flow measurement had been evolved and in compliment to its originator officially changed the name to "The Parshall Flume." This device has now come into wide use in the West in irrigation districts, where such flumes are installed with throat widths of 1 to 40 ft., and in corresponding maximum capacities of 0.5 to 2 600 cu. ft. per second, or approximately 0.3 to 1 700 m.g.d. The measuring range for "free flow" discharge, *i. e.* using the single gage heights, averages about 15 to 1. An important characteristic also is the relatively low loss of head, about one quarter that of the usual weir.

At the Providence plant, an open concrete chamber had been constructed to receive the flow from the two main effluent channels just preceding entrance to the tunnels leading to the Bay. In this chamber it was decided to construct a Parshall Flume to be suitable for a future possible flow rate of 120 m.g.d. The throat width adopted was 10 ft., and correct proportions for the Flume dimensions were strictly followed. While the Parshall Flume is said to be largely independent of approach conditions it was realized that the curved entrance of the two influent channels would result in considerable turbulence, and it was decided to take off two pressures, one from each side of the entrance cone at the specified location to produce an average height in the common float well. Brass pipes, 2½-in. in diameter, separately valved, lead from these vents to the float well of the recording instrument. An excellent, smooth job of reinforced concrete work was done in constructing the Flume. At the sill of the throat, where the velocity is highest, a section of steel rail was embedded. Manholes were located over the pressure-valve connections at the opposite sides of the inlet cone, and suitable connections were provided for back-flushing the pressure pipes. The Flume itself, due to its velocity-accelerating characteristics, is "self-scouring."

For the recording instrument a Builders Iron Foundry Type Y Register-Indicator-Recorder, specially modified, was used. It is housed in a neat building of selected faced brick, having floor dimensions 6 ft. 7 in.

by 4 ft. 2 in. at the head of the flume chamber. This instrument is operated by a 24-in. diameter float in a concrete well immediately beneath the house, the well being connected, as stated, to the two flume vents. The connection between float and register is a flat monel ribbon, properly counterweighted, and perforated to fit over pins in the register-actuating pulley. There are the usual three dials: rate indicating, rate recording and totalizing. The range of graduation is from 10 to 130 m.g.d.; the corresponding float travel is from 0.5 to 2.7 ft. The curve of this relation is quite flat. An additional feature is a Selsyn motor transmitting-attachment which actuates a secondary rate-indicating dial located in a basement window of the distant Chemical Laboratory, convenient for regular observation by the plant operators.

The Providence Parshall Flume has now been in continual and satisfactory operation for eight months. Acknowledgment should be made to Julius W. Bugbee, chemist in charge, for many details of design, and to Messrs. Waterman, Nolan and Kendrick of the Public Works Department for helpful coöperation. In a sense the installation represents engineering work of a pioneering nature, since no flume of this type approaching it in size has been heretofore utilized for measuring sewage. The Flume has shown itself to be sensitively responsive to small changes in rate of flow. The variation in flow thus far recorded is from 5 to 120 m.g.d., the latter occurring recently during a heavy storm. At extreme high rates it has been noted that the turbulence in the converging section is materially less than at a 50-m.g.d. rate.

Mr. Parshall, in expressing his great interest in the installation, remarks that while it would seem that the flow is very much disturbed, he yet feels certain that any resulting inaccuracy "would be well within 5 per cent." While the recorded rates since installation have been entirely consistent with rates obtained from carefully kept weir records under similar conditions in previous years, it has not been possible, until very lately, to make comparisons with other measuring devices, such as weirs and current meters. All of these have indicated accurate Flume performance. During the week of September 11, Mr. Bugbee, by altering the course of the sewage through certain channels, was able to get a direct comparison with a 10-ft. standard weir, and he reports the following gratifying results:

Date of Test	Duration of Test	Average Flow m.g.d.		Percentage Difference
		Flume	Weir	
Sept. 13	9:45 A.M. to 12:45 P.M.	48.00	47.91	0.19
Sept. 13	12:45 P.M. to 1:45 P.M.	23.00	22.94	0.26

It will be noted that agreement is within approximately ¼ per cent.

Apparently the Parshall Flume should become increasingly useful for the measurement of water and sewage in open channels.

MODERN WATER CHLORINATION PRACTICE.

L. H. ENSLOW.*

[*Read September* 21, 1933.]

Chlorination of potable water supply dates back to 1908 in America but was practiced successfully as early as 1902 in Belgium. For a long time considered, in conjunction with filtration, as being the line of last defense only, chlorination is now an acknowledged integral part of the complete process—and an increasingly important part.

Interestingly enough, in early chlorine applications in conjunction with filtration the chlorine was applied prior to filtration, only to be changed to the filtered water. In numerous instances, it has now been returned to the raw water, or the coagulated water, to produce desired effects in filtration practice other than mere disinfection.

Although chlorination of water had become fairly widely established in America by 1915 it was not until 1918 or 1919 that chlorination was placed on a basis subject to prompt and simplified control. I refer to the practical application of the orthotolidin test in the control of chlorine dosages so as to insure the presence of residual chlorine in the treated water under all conditions—thereby insuring adequacy of disinfection without over chlorination—an accomplishment which previously could only be assured through tedious bacteriological examinations, requiring several days for completing the test. Such time-requiring control technic frequently had proved inadequate for continuous protection. Chlorination having been placed on a dependable basis by employing this simplified control method, the process gained markedly in favor because of its greater dependability and reduced likelihood of over-chlorination resulting in inevitable taste complaints.

Combating Taste Troubles.

When dealing with fairly pure waters the taste problem was solved by resorting to the use of this new and handy index of chlorination efficiency. But to make matters interesting to the water purification man, along came the problem of taste production which was traced to the reactions between chlorine and foreign matter in the water—in many instances associated with human or industrial pollution or pipe coatings, but in others created by natural conditions, notably the presence of decayed vegetation and disintegrating algae.

Super and Dechlorination. To combat such disturbing conditions it became necessary to open up a new bag of tricks in chlorination practice

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which included the introduction of super-chlorination to destroy, by oxidation, the offending materials and thereafter employing dechlorination to obviate chlorinous tastes due to the excess chlorine added. This procedure has been effective but somewhat expensive.

Permanganate Treatment. The application of permanganates in conjunction with ordinary chlorination was a cure in many instances and continues to be effective in dealing with certain conditions wherein algae and decayed organic matter have been responsible for the production of color, tastes, odors, hydrogen sulphide and ferrous iron or manganous manganese. Naturally, permanganates are most satisfactorily applied to the water prior to filtration. This treatment is most effective when used in combination with prechlorination which insures removal of the objectionable manganese during the coagulation and filtration process. One should not overlook the value of permanganates in solving future problems. Even though other newer processes appear to have a greater appeal on the whole there are situations in which permanganate treatment in conjunction with prechlorination has distinctive advantages as an oxidizing process.

Prechlorination. Repeated warnings that prechlorination would destroy the efficiency of the bacteria removing "Schmutzdecke" of the filter beds were disregarded by a few hardy operators who were of the opinion that a stiff application of so powerful an agent as chlorine might be effective in keeping down obnoxious growths and putrefaction in the basins and filters and serve moreover as an oxidizing agent to destroy many of the end products of decay or the essential oils already present in the raw water. Half hearted attempts in this direction frequently failed. But they, who in the words of Sir Alexander Houston, showed "chlorine boldness" in place of "chlorine temerity", were successful for the most part; if not immediately then eventually so. The process in reality amounted to moderate super-chlorination and gradual dechlorination through progressive conversion of much of the residual chlorine into nascent oxygen, the remaining excess being removed during passage through the filter beds. In addition to taste improvement it was apparent to many that other technical and also economic advantages, such as improved coagulation, reduced filter bed troubles, lessened wash water requirements and decreased putrefaction of sludge in the basins, were being realized. The move toward heavy prechlorination was decidedly in the correct direction and has lead recently to the return in several instances to heavy chlorination of the raw water and discontinuance of pre-ammoniation. Because ammoniation slows down the oxidizing action of chlorine and thereby its destructive action on organic impurities — including those responsible for objectionable tastes — as well as for other reasons, ammonia may be more effectively applied to the filtered water.

For the work of taste *destruction*, raw water chlorination preceded or followed by applications of activated carbon is preëminently the most uni-

versally successful procedure. More will be said later about this process and the efficacy of post-ammoniation.

The addition of ammonia to raw waters ahead of chlorination has been in several instances an effective taste-prevention procedure but care in manipulation is required, and the practicability of the process in comparison with postammoniation is to be questioned.

Postammoniation. Ammoniation was first practiced for taste prevention by Houston in England and later by McAnnis at Greenville, Tenn., in 1926 where the process proved, according to the latter, to have been effective and "as simple as rolling off a log." From this beginning the use of ammoniation in conjunction with chlorination has grown until there are now 650 installations of ammoniation equipment in America.

At Greenville it was a case of preventing chlorinous tastes and more especially tastes which residual chlorine produced when coming in contact with the lining of the mains or the painted interior walls of the standpipe. To many the process was equally as simple as at Greenville, but to others it was somewhat disappointing, although the chlorine required to maintain a definite residual chlorine value in the treated water could be and was reduced in practically every instance. The reason for this was readily explainable. The chlorine fixed as one or the other of the two chloramines (principally dichloramine in waters of less than pH 7 and almost wholly mono-chloramine in waters with as high as pH 8) had a much lowered oxidation potential. Hence it did not attack and combine with reducing organic products as rapidly or as pronouncedly as did the more powerful chlorine or the hypochlorous acid which chlorine produces. Neither did chloramine-chlorine attack the living organisms as vigorously. This constituted its one weakness.

The chloramines, being mild compounds, did not impart a chlorinous taste to very pure waters nor did they combine with foreign bodies to produce tastes in less pure waters. It merely became a matter of securing the most efficient distribution of the ammonia prior to an efficient and thorough distribution of the chlorine to demonstrate the true value of the process and to produce maximum economies. We have long since learned that the more perfect the distribution of these two chemicals the less the quantities of chemicals required. This is particularly of major importance in reducing to a minimum the requirement of the more expensive ammonia. Modern practice involves the use of lower and lower ratios of ammonia to chlorine. Prior to an understanding of the importance of distribution and mixing, ratios of 1 ammonia to 2 chlorine, or even higher, were employed — whereas, now, in postammoniation practice (ammonia added after filtration but prior to final chlorination) the ratio of 1 ammonia to 4 chlorine proves to be ample in most instances, and even lower ratios are effective in many cases.

The required ammonia-chlorine ratio is a fixed quantity for any fixed condition. It is decidedly variable however with varying pH values of

waters, and if we assume the pH value to be adjusted to a constant point, as is done in modern corrosion control, then the quantity of ammonia to be added will be governed by the *available* ammonia content originally present in the water treated. In other words there is a definite minimum of ammonia needed to produce chloramines from a given weight of chlorine. If there already is some ammonia present in the water, it becomes necessary to make up only the deficiency. It may not be out of order then to suggest the application of lime in pH adjustment prior to postchlorination or along with the chlorine via a common hose line. In doing so it becomes necessary, however, to recall that in waters adjusted to pH 8 or higher the reaction velocity of the monochloramine produced is materially slower than that of dichloramine. Thus the contact period required for efficient bacterial destruction must be increased unless prechlorination has been employed. If prechlorination, producing maximum safety, has been employed, the major value of the ammonia-chlorine post treatment becomes that of precluding tastes and insuring the several advantages of sterile conditions in mains, reservoirs, dead ends and a safeguard against chance pollution from unforeseen causes, including leaky cross-connections.

Returning to the matter of the mild behavior of chloramine chlorine it has been learned, as was to be expected, that chlorine in this form created a saving in the amount of chlorine necessary to secure the same residual chlorine value found effective before beginning ammoniation. It was not appreciated until later, however, that the milder form of chlorine being a less efficient bactericide a high concentration would be required in safe practice, unless the water had already been effectively prechlorinated. That an increased contact period was necessary, was generally appreciated. On the other hand, the considerably slower bacterial attack at pH values above 7.8 and at temperatures under 10° C. was not accorded sufficient recognition until studies made by Gerstein in the Water Control Division of Chicago and by Holwerda in Holland revealed the importance of these two factors.

In discussing the Chicago report in 1931, the writer felt it necessary to sound a note of warning to operators of chlorinating plants employing ammonia and saving chlorine by maintaining no higher residuals than had been found requisite without ammoniation. Subsequently this warning has been proved a justifiable one—one to the effect that a new and lesser weight must be given to residual chloramine chlorine values. Further, that it would be found that higher residuals would be required in many instances. How much higher, would be determined by such factors as temperature, alkalinity and the available period of contact. It can now be stated with fair assurance that about double the residual to produce former efficiencies with ordinary chlorination (after 10 minutes of contact) will be required when changing over to the ammonia-chlorine treatment. That 20 minutes of contact and preferably a longer period should be anticipated as the minimum safe requirement. With waters as alkaline as pH 8.0 and higher, there is a hazard in making any sort of rule with reference to safe practice

in cold weather. In every instance the operator should determine safe practice for the conditions in hand. It is fortunate that much higher residuals may be carried without complaint from taste. This makes it possible to start at high figures and drop the dosage gradually to a point where further lowering will constitute doubtful safe practice. This rule applies equally in the control of swimming pool chlorination as in that of potable water.

MODERN AMMONIATION.

In modern practice, ammoniation is being employed to do two major things: (*a*) to prevent chlorine tastes created by over-chlorination and (*b*) to create a persistent form of available residual chlorine which is of pronounced value in the distribution system.

Concerning the latter, I refer to the fact that the presence of the chloramine keeps down dead end complaints (red water or black water) and reduces the necessity of frequent flushing of mains. It controls growths of *crenothryx* and the like in the mains and suppresses biologically created tuberculation in long mains or in distant fringes of the distribution pipe system. It prevents putrefaction of pipe slime carried to the dead ends, the latter being a breeding ground for acid-producing organisms that sap the oxygen from the surrounding water and cause the production of foul odors, including hydrogen sulphide, and vile tastes, including phenolic substances in dead-end water, not to mention high carbon dioxide and iron content due to corrosion. It is a fortunate water-works superintendent who does not receive numerous summer complaints from consumers on dead ends or who has not tasted of the iodoform water at such points when chlorine alone has been used and the residual chlorine in the fresh (new) water mingles with the foul water stagnating in low-velocity sections of the system.

To those in charge of unfiltered supplies which are subjected to variable flow or pumping rate or variable composition of the water, or both, wherein the water flows directly to the system or reservoir floating on the line, the ammoniation process has been a great boon. The careful control of chlorine dosage in such cases has become of secondary importance. The upper limits of non-taste-producing residual in the form of chloramine are sufficiently high to permit setting the minimum chlorine requirement frequently for maximum rates of draught without having the maximum residual at minimum draught or flow exceed that permissible. The early morning "chlorine cocktail" is a rare beverage in the consumer's home served with water treated by the ammonia-chlorine process which is accorded some semblance of supervision.

Referring now to the weakness of the process, *i.e.* the longer contact period required or the higher residuals requisite with shorter or longer contact, the process can be reversed to procure the previously outlined advantages, when applied to waters which do not produce tastes other than

those of chlorine, *i.e.* no chloro-byproduct tastes. If the available contact period is short between the points of chlorination and consumption, it may be more feasible to insure rapid disinfection by adding the chlorine first and applying the ammonia thereafter to convert the residual chlorine to chloramine. In other words, add sufficient chlorine to leave 0.2 to 0.4 p.p.m. residual after a 10 minute or longer contact and then apply ammonia and simultaneously the alkali, if such is used to raise the pH value to control corrosion. One interesting feature of this reversed procedure is the reduction in ammonia requirement. Only sufficient ammonia is needed to combine with the residual chlorine and not an amount required to satisfy the entire chlorine dosage in accordance with usual practice. Economically and hygenically such a procedure has its distinctive appeal where practical.

Activated Carbon and Prechlorination.

By all odds the most modern and reliable practice in taste control involves the use of powdered activated carbon in conjunction with prechlorination or without it. From all that can be learned by observation, however, a combination of prechlorination and carbon treatment produces results superior to those obtained by the use of either alone. Depending upon the desired accomplishments, the carbon is applied to the raw water along with or ahead of the chlorine or it is applied to the coagulated settled water just ahead of the filters. The best practice under variable conditions is subject matter for a paper in itself. For assisting coagulation and suppressing sludge putrefaction in addition to taste removal, or where short-period coagulation basins are involved, it seems sufficient and preferable to apply carbon to the raw water. If the water is turbid or the coagulation basins large, no carbon is added to the raw water, only to the clarified water. There are intermediate conditions wherein application at one point at one time and at another or both points at other times is apparently best practice.

The great value of carbon is its ability to remove colloids from the water. This means actually increased purification of the water and with it taste *removal*, in distinction to taste prevention. Indicative of the actual adsorption and removal of chlorine-demanding substances from the water is the observation that water (raw or settled) when treated with powdered carbon prior to chlorination has its chlorine-demand value reduced—sometimes appreciably so. There is evidence to the effect that a small quantity of powdered carbon added to the raw water improves coagulation. Interestingly enough in such high dilution the carbon does not dechlorinate the water. To the contrary, in combination with carbon, the residual chlorine after passage through the basins becomes more pronounced, but when powdered carbon is laid down on the sand filter bed it completely declorinates the water passing through it and simultaneously adsorbs and removes taste-producing chloro-products that may have been formed together with

other taste-imparting elements. That is to say, powdered carbon when dispersed, as is the case in water pre-treatment, passes through chlorinated water without combining with the diluted chlorine, but carbon in compact form as on a filter bed or that in granulated carbon units does react with the chlorine to remove it completely.

Experience indicates that probably the greatest overall economy and efficacy result from the use of mild superchlorination of the raw water, involving dosages of between one and three parts per million. These improve coagulation, oxidize organic constituents, control algae and thereafter dissipate to a marked degree in passing through the basins. Furthermore activated carbon applied to the filter influent completely removes the residual chlorine and along with it collodial constituents including taste- or odor-producing bodies.

Filters of granular carbon serve the same purpose, and there is at least one installation of such filters for the purpose of dechlorinating a clear but colored water which has been bleached by super-chlorination and contact ahead of the carbon units. Without prechlorination, however, carbon beds have their troubles and it appears that chlorine is a most satisfactory material to keep activated carbon active if not actually render it more active.

Some Important Aspects of Prechlorination.

Prechlorination practice has expanded considerably in recent years. Possibly this has been the result of the fact that seeing is believing when it comes to benefits derived, possibly it has been due to the studies conducted by the U. S. Public Health Service at the Cincinnati Station which are given a favorable report as to the efficacy of the process (excluding bacterial efficiencies of the filters), and possibly it is the result of the increasing use of activated carbon and also the better condition of the filter units wherever prechlorination is employed.

In some instances prechlorination has been less costly than post-chlorination alone. Why? Because the use of more chlorine has made it feasible and practical for the consumer to purchase chlorine in ton containers at a much more favorable price. As a result, the total consumption of chlorine costs no more and probably less than previously, notwithstanding the increased use.

Not the least advantage to be secured from prechlorination has been the high safety factor which the process provides. A purification plant running on prechlorination has at all times a tremendous barrier between the raw water supply and the consumer, in the nature of sterile filters and coagulation basins filled with highly chlorinated water. Lapses of chlorination or dependence on postchlorination, faulty contact periods and reduced efficiencies when ammonia is employed in postchlorination become matters of secondary concern so long as prechlorination is effectively accomplished.

Recent experience at Detroit, in which prechlorination prevented possible difficulties when postchlorine dosage control tests were vitiated by misleading readings of residuals, is a case at point.

An Ideal Plant Operating Arrangement as the Author Views the Situation.

It is probably hazardous to set forth one's views in regard to ideal filter-plant operation at this time of rapid improvements in such matters, but the author can not help being impressed with the highly successful accomplishments at certain plants, among them that at Richmond, Va., at which the following procedure is involved.

A. Prechlorination (12 to 25 lb. per m.g.; residual of 1 p.p.m. or more).
B. Adjustment of pH in coagulation control with lime or acid when required.
C. Coagulation by quick-mix followed by mechanical flocculation.
D. Sedimentation.
**E*. Powdered activated carbon applied to filter influent.
F. Ammoniation and postchlorination.
G. Adjustment of pH to 8.4 with lime.
H. Residual chlorine of not less than 0.2 p.p.m. found at any point in the distribution system—no complaints.

*Note: The long period of sedimentation following coagulation has rendered the use of raw-water carbon treatment of greater expense than its use at the filters in the Richmond plant. At other points carbon application to the raw water alone or split between the raw and applied water may prove more effective.

The new plant of the Lehigh Water Company at Easton, Pa., has with success recently modeled its treatment on the same pattern as indicated above and doubtlessly there are others which have had similar experience. It is the author's belief that the Richmond plant was the first to completely work out this scheme as the result of having tried variations of many kinds—including preammoniation with prechlorination.

Some Incidental Uses of Chlorine.

The use of chlorine and hypochlorites for disinfection of new mains has been thoroughly dealt with before this Association during the past few years. The process is becoming increasingly popular.

For oxidation of manganese and iron in raw waters, chlorination is being more widely used than ever before.

For cleaning filters, superchlorination of the sand beds appears to be the most satisfactory scheme evolved. In superchlorinating filter beds, the practice is to drain the sand bed down to the gravel and while filling with influent water, dropping over the troughs, chlorine gas is diffused into the

inflowing water through large porous diffusors or chlorine water is applied from a regular chlorinator, or an injector of hard rubber, or other resistant material. The bed is allowed to remain filled to just above the sand surface with the chlorine water for 18 to 24 hours and is then vigorously stirred on the surface and washed thoroughly. The treatment is repeated until the sand becomes clean. Once clean the treatment is repeated at intervals. As a ready substitute for chlorine, strong solutions of hypochlorites are effective and simple of application. The secret of filter bed cleansing lies in the destruction of living growths and the granulation of organic films and mud around the sand grains and gravel. The resulting sharp clean sand will not create mud balls, and cracks and bed shrinkage away from the side walls are done away with. The sand in filters subjected to a series of systematic treatments looks like new sand. The cost of sand removal, washing, or replacement has been saved.

A new scheme of coagulant production involves the passage of chlorine water upward through a tower filled with scrap iron. The overflow water contains ferric chloride in weak chlorine water thus producing at one operation coagulation and prechlorination. This scheme is similar to that developed at Baltimore, Md., for the production of chloride of copper in algae control in which the dosage of copper is a function of the rate of chlorine water application at the bottom of the tower.

At certain plants chlorinated ferrous sulphate "chlorinated copperas" continues to be used as the coagulant best suited for the conditions at hand. Lately, ferric sulphate has become available at prices competitive with copperas and chlorine and renders this new material attractive because of its ease of handling and efficiency.

The Handy Package of Chlorine. One could hardly discuss modern practice in water chlorination without referring to the handy package of chlorine which is represented by the dry concentrated hypochlorites: "H.T.H." and "Perchloron." These products contain 62 to 75 per cent. of available chlorine, and are now widely used for incidental or emergency chlorination, cleaning filters, sterilization of mains, algae control, swimming pools, small water supplies and large reservoirs.

Discussion.

Warren J. Scott.* It seems to me that Mr. Enslow's review of modern water chlorination practice is an excellent one and contains considerable information that is worthy of the study of waterworks men engaged in the every day practice of chlorination. It does not appear out of place to say that the author of this paper deserves commendation from the waterworks fraternity for the quality of the research work which he has carried out on this subject as an agent of the Chlorine Institute and for his promotion of coördination of various research agencies in this field throughout the United States.

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Our New England problems with chlorination are somewhat more restricted in scope than those in many other parts of the country, where raw waters are frequently highly polluted and turbid. With comparatively few exceptions, chlorination in New England is used mainly as a preventive against chance contamination of waters which are derived from relatively clean watersheds. As an example of conditions which are probably somewhat representative of New England, approximately 77 per cent. of the water supplied through public water supplies in Connecticut are chlorinated. Of these 77 per cent., only about 12 per cent. are filtered. Another estimated 15 per cent. are filtered but not chlorinated.

One point which Mr. Enslow has not discussed at length but which has been discussed at previous meetings, is the presence of interfering substances that may cause a false reading for residual chlorine by the ortho-tolidin test. Manganese in the manganic form causes a yellow color with ortho-tolidin, similar to that obtained with chlorine, as does iron in large quantities, and nitrites in fairly large quantities. We have had some trouble with the presence of manganese in some of our Connecticut supplies. The Connecticut State Department of Health has forwarded mimeographed information to waterworks officials on this subject and recommends that occasional checks be made for the presence of manganese.

The ammonia-chlorine method of disinfection is attracting much interest on the part of waterworks superintendents and has accomplished good results in many instances. It is being successfully employed at Bridgeport, Stamford and South Manchester. One great disadvantage on most of our Connecticut supplies, as mentioned by Mr. Enslow in his paper, is the short period of contact which exists on many of our systems between the points of application of chemical and the taps of consumers. There is some uncertainty as to what contact period is necessary for adequate disinfection. In Chicago, it was found that for the same chlorine dosage, the bactericidal velocity of ammonia-chlorine treatment is decidedly less during the first two hours after the treatment is applied than that obtained with the chlorine alone. The larger the amount of ammonia present, the greater is the lag of bactericidal effect. We have generally considered that a minimum contact period of 30 minutes should be provided.

I think it is well for us to proceed cautiously so as not to let down the bars and permit inadequately disinfected water to reach consumers, even though other advantages of the ammonia-chlorine treatment may be numerous. I think probably in many instances where contact periods are short, we will have to forget the benefits of taste elimination and add the chlorine considerably ahead of the ammonia. This will give adequate disinfection, improve bacterial conditions throughout the distribution system and help in the elimination of dead-end troubles, even though the chlorine may form in some cases rather objectionable chlorinous tastes that might be eliminated by reversing the order of application of the chlorine and ammonia.

It is somewhat difficult under the Connecticut conditions to measure operating results of the ammonia-chlorine treatment in the field as to the effect of the bactericidal lag upon water supplied to nearby consumers, even though samples are neutralized with sodium thiosulphate immediately upon collection so as to cut off the effect of the disinfectant during transportation to the laboratory. This is because many of our supplies show organisms of the coli-aerogenes group in 10 c.c. rather infrequently in the raw water. While it can undoubtedly be done over a long period of time, it is not so easy to measure the results on any one particular supply under such conditions as would be the case with polluted waters. With the increasing use of chloramines in water disinfection, I am wondering if some consideration should not be given to the establishment of the use of sodium thiosulphate for immediate neutralization of the disinfectant at the time of collection of sample as a routine procedure for laboratory examination, so that we may be certain that the analysis represents a true picture of the water as delivered to the consumer. I know that in the case of disinfected sewage samples, we have found that chlorinated samples even with contact periods of thirty minutes, have shown much poorer analytical results with neutralization than without.

The author of the paper has mentioned the lag in bactericidal velocities when the water has a high pH. There are very few of our Connecticut supplies that have a pH value above 7.8.

E. SHERMAN CHASE.* One thing that Mr. Enslow mentioned to which I take slight exception is the control of chlorination through the orthotolidine test as being the first scientific method of control. Prior to the introduction of orthotolidine as a control agent we were using the starch iodide test. We controlled the application of chlorine to the raw Hudson River water by the use of the starch iodide test, which was a crude test, not as sensitive as the orthotolidine test as it has now been developed, but in those days that was the best we could do. Furthermore, during the Great War it was somewhat difficult to secure the proper brand of orthotolidine.

It is also interesting to note in Mr. Enslow's paper the emphasis placed upon the refinement of water by chlorine and allied treatments as contrasted with the hygienic control of the water by chlorine treatment twenty years ago. I think I am correct in stating that the first, or one of the first water supplies to receive chlorine treatment, was here in the state of Connecticut, at Stamford. That was in 1913, exactly twenty years ago. It is quite fitting that this ASSOCIATION should have a paper on chlorination here in Connecticut on its twentieth anniversary.

Referring to the activated carbon treatment of water, I would like to ask if in the United States activated carbon has been applied directly to reservoirs. I have read of some cases in England where a water supply reservoir has been treated with activated carbon.

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*Of Metcalf and Eddy, Consulting Engineers, Boston, Mass.

FRED E. STUART.* I can outline the experience of Belmont, Pennsylvania. The supply was troubled with algae growths which were combated by copper sulphate. The taste was so objectionable that the water could not be used. Powdered activated carbon was employed in the same manner as at Bath, England, in concentrations of 4.5 p.p.m. This was applied after the addition of the copper sulphate. The water close to the intake was first treated, and when it was found that the taste was greatly improved they finally treated the whole reservoir. It is interesting to know that activated carbon treatment can be used in conjunction with copper sulphate, and that would be the way it would be recommended.

EARLE DEVENDORF.† Mr. Enslow's paper I think is a very timely one, and one that has been given a great deal of thought and is based on a great deal of experience and contact.

Chloramine treatment in New York State has been very interesting. Some of the results we have been able to obtain have been phenomenal.

One of the first places in New York State that employed chloramine treatment did so primarily to control the objectionable tastes due to phenols. Experience indicates that for phenol taste prevention it is necessary to put in the ammonia before the chlorine, whereas in other cases, where treatment is just for continuing the sterilization effect, it does not make much difference, apparently, whether the chlorine is added ahead of the ammonia or the reverse.

I should like to ask Mr. Enslow as to his experience in making the orthotolidine test. If the sample is collected ten minutes after dosage, or presumably shortly after the application of the chemical, as against collection of the sample after thirty minutes or an hour's contact, what is the experience as to the true conditions of the supply? For instance, in swimming pool control where chloramine was used we require that a predetermined amount of thiosulphate be added to the sample so that immediately upon collection the excess of chlorine is dissipated in order to determine the bacterial quality of the sample at the time of collection rather than the quality, after some longer contact with the chloramine. What is the proper procedure in collection of samples?

MR. ENSLOW. It depends on how much residual chlorine you want to carry into the system. Realizing that the use of the ammonia cuts down the rate of dissipation of the chlorine, you can see very clearly that a lot depends on how many minutes elapse between the point of application of the chlorine and the ammonia before you make the orthotolidine test. It is not like straight chlorination, which acts very quickly, in which event you get the same result after ten minutes as you will after forty, for example. Furthermore, you will find that there is an absorption of the chloramine chlorine even out in the system, due to its absorption by the organic matter which is originally in the water and on the inside of the pipe system. That

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†New York State Department of Health, Albany, N. Y.

is especially true on dead ends. So that I would say that it is a matter of determining at the control point what residual one must carry in order to get a desired residual out in the system, and I do not know of any way of doing that except by the cut and try method; and, to be safe, starting at a high value and working down to the most economical point which is safe in practical operation.

MR. DEVENDORF. We use thirty minutes now. Are we right?

MR. ENSLOW. I think that is good practice. I would not be sure that I would not test again after an hour or two, just to make certain that I still got the same residual. The question might have been, how much contact period one should allow after adding orthotolidine — an interesting question.

MR. DEVENDORF. Answer that, please.

MR. ENSLOW. Detroit got into trouble because they followed Standard Methods in making the orthotolidine test on ammonia-chlorine treated water. I do not wish to criticize Standard Methods, but I must call your attention to a condition met in Detroit. They followed Standard Methods, allowing thirty minutes after adding the orthotolidine before making their comparison. They actually found though, that the color reached a maximum much before that period and then faded out gradually. Why was that? That is simply answered, when you understand that when adding the acidic orthotolidine solution you are lowering the pH value, and you do not get the color reaction until you get a pH of about 2. When you get the pH down to 2 you are producing nitrogen trichloride (trichloramine) from either dichloramine or monochloramine. You can hold the chloramines by raising the pH value, but in doing that you lose some of the available chlorine. Going from the high pH to one lower, you do not seem to lose available chlorine, you do convert it to a more aggressive form, and you get an agent which attacks some products which the dichloramine or monochloramine does not attack. That means, too, an increasing color production for a while after adding the orthotolidine, which reaches a maximum after some lapsed time and then the chlorine being gradually dissipated the yellow color drops off.

My candid opinion is — and I am going to stick to it until somebody proves I am wrong — that the maximum color that one gets after adding the orthotolidine, whether it is two minutes or five minutes or ten minutes, is the most nearly correct reading to take.

When using the ammonia-chlorine treatment, it is true that in cold weather there is a considerable lag in the maximum production of color with orthotolidine. It may run as much as fifteen minutes, but if you wait longer than fifteen minutes you are likely to get most any thing, which is capable of reaction, with orthotolidine. My experience has been such that I would rather use ten minutes than fifteen. If the maximum appears at five minutes I would take it. The effect is more pronounced in sewage testing than in water — this, because there is more organic matter in sewage.

In answer to the other question, I would say that I would allow a contact period of thirty minutes before putting the orthotolidine in, and then allow sufficient time to be sure of securing the maximum color development up to fifteen minutes or possibly longer if the water sample is very low in temperature—say 10° F. or less.

A MEMBER. In letting the sample stand, is there any possibility at all that the ammonia would undergo slow oxidation and form nitrites?

MR. ENSLOW. No. Nitrites are formed from the ammonia added, almost entirely by biological action of nitrifying bacteria. Particularly is this noted when ammonia is added ahead of non-sterile filters—and, out in dead ends where the chlorine has been exhausted.

The MEMBER. You feel it is impossible to do that by chemical means?

MR. ENSLOW. I would not want to make a flat-footed statement of that kind without looking further into the matter. I will say this: That there have been some reports that nitrites seem to be built up in the distribution system when using ammonia-chlorine treatment. Whether or not that might be due to the slow oxidation of ammonia by the chloramine or hypochlorous acid, I hesitate to say. As a matter of fact, I think there is a bit of misleading information on that point. I am not at all sure the false color, if any, is due to nitrites, as some believe. This, for the reason that in taking samples of activated sludge effluent, and other effluents, it is remarkable how much nitrite nitrogen you can have and not get any color with orthotolidine. In other words, if nitrites in reasonable quantities had interfered with orthotolidine in practice we would have been in difficulty with control of sewage chlorination some time ago. I for one can't think that it is nitrites. Not that nitrites are not present when you make the test, but that is far from proof that the nitrites are responsible. I think it is really a break-down product from the ammonia-chlorine compound that gives you this interference, because it is a fact that some of these compounds are very slow to produce color with orthotolidine. Further, the nitrite reaction gives a brownish color and chlorine a greenish yellow.

MR. SCOTT. Has any research work been undertaken recently, or is there any research work under way now to develop any reagent other than orthotolidine that is specific to chlorine without being affected otherwise?

MR. ENSLOW. During the last five years there have been a number of attempts to discover an indicator which would replace orthotolidine, but to date there is none that has proved satisfactory. Every indicator that one would use apparently depends upon the oxidizing effect. There is one basic principle involved, and that is that you have a colorless compound and you oxidize it and produce a colored compound, and that is why permanganate, manganate, manganese, and high nitrites all interfere. Hydrogen peroxide will do it when present in high enough concentration. A study is going on now at the New Jersey State Department of Health laboratories which involves the use of methyl red, in which a standard methyl red solution is dropped into the sample of chlorinated water. The

concentration of chlorine is a function of the quantity of the red dye the sample will bleach to colorless. The attempt is an interesting one but when it comes to determination of chloramine-chlorine I fear there are difficulties to be met. Any one who wants to do a real favor for this profession should evolve or suggest a test which will prove specific for chloramine or chlorine, and when he does that all the chemistry text books will have to be rewritten.

FRANCIS H. KINGSBURY.* It occurs to me that perhaps from a bacteriological control and safety point of view thirty minutes might be on the safe side. From a control of the quantity of chemical used perhaps maximum color might be obtained earlier, but wouldn't the standard of thirty minutes be a safer procedure to one who was not supposed to maintain fine control of the chemical?

MR. ENSLOW. I think it would not be unsafe. It was unsafe in the case of Detroit. You see there the thirty-minute reading was higher than the fifteen-minute reading.

MR. KINGSBURY. Higher color?

MR. ENSLOW. Yes. That meant that they were going under the false assumption that they had a certain residual there and they did not have it.

MR. KINGSBURY. Didn't you take the highest color, whether it was ten minutes or fifteen?

MR. ENSLOW. But I did not go as high as thirty minutes. I said, up to fifteen minutes. I am not sure I would go that high for safest practice in chlorine control.

MR. KINGSBURY. Then there appear to be two peaks in the color?

MR. ENSLOW. That is right.

MR. KINGSBURY. One would indicate chlorine, and the second peak, which might be higher, would indicate something else?

MR. ENSLOW. Yes, that is right but is rarely encountered except where manganic or ferric compounds are present in an organic water or sewage.

GORDON M. FAIR.† Mr. Devendorf mentioned the application of the chloramine process to swimming pools. I would like to ask Mr. Enslow whether he thinks the use of chloramine in swimming pools is here to stay. As I see the matter, what we are attempting to do with swimming pools is to disinfect whatever materials are discharged from one person before they reach the next person.

MR. ENSLOW. I believe that the ammonia-chlorine treatment of swimming pools is very much here to stay. First, it makes a very much more pleasant swimming pool to use if the ammonia-chlorine treatment is employed, and people will use the pool more than otherwise. I have had experience in that direction, and I know that there were a lot of complaints and little use of swimming pools when chlorine alone was used, whereas in going over to the ammonia-chlorine treatment the patronage increased rapidly.

Secondly, I believe from all I have seen that it is certainly possible to

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carry a higher residual chlorine with equal effect. You see, this is all a matter of temperature, mass reaction, nothing else. As I see it, the rate of fall is a function of the oxidative effect or potential—possibly the dissociation factor. If 0.5 p.p.m. of residual chlorine will do the work with ordinary chlorination, probably 1 p.p.m. of residual will do it with the ammonia-chlorine treatment. That is about a 2 to 1 ratio in the residuals required for comparable disinfection in short periods. The swimming pool that is used by few people is not a dangerous pool, and never was. But where you get a large concentration of bathers, when you are using ordinary chlorination it is not very long until all protection is gone. You can't start a pool in the morning with enough residual chlorine in it, and get people to go into it, to be effective during the entire day of use. Ordinary chlorine residual is used up because of its activity. When one forgets the little things and looks at the fundamentals of this problem of pool sanitation the advantage of the ammonia-chlorine treatment in swimming pools becomes better appreciated.

E. T. CRANCH.* Would you consider it safe, in view of what Mr. Scott says, to attempt to use ammonia in a supply that is not filtered, in a case where the consumers are attached to a line within 200 feet of the pumping station? Is it possible to raise the dose to a sufficient amount to insure a supply of acceptable water and still get the benefit of the ammonia?

MR. ENSLOW. I would answer, "No," on that, just to be safe. Here is the case in Chicago, for example. Chicago turned down the ammonia-chlorine treatment two years ago (1931) absolutely on the basis of the fact that it took an hour and a half or so to get effective disinfection. You know, in Chicago they put the chlorine in and then drink it right now, chlorine and all, I mean. But with a study of these problems frequently you can find a way out. Chicago's way out has been this: They have gone way out in the lake to the intake crib, and are putting the ammonia-chlorine treatment in at that point, in order to avail themselves of the contact period in the tunnel between the intake crib and the shore line. They have gone to that after at one time having just thrown up their hands saying, "No, we can't possibly use the ammonia-chlorine treatment." At times a thing may look very difficult, but as you give it a little study it may work out. My candid opinion is that there is going to be a good deal in the way of adapting the ammonia-chlorine process by putting in the chlorine first and then the ammonia. Of course that can not be done if the water contains impurities to produce byproduct tastes.

SAMUEL M. ELLSWORTH.† There is one point that Mr. Enslow brought out in his paper which I think might be emphazised. In regard to the ammonia-chlorine treatment, it is possible to use larger amounts of chlorine and still obtain proper disinfection without the resulting bad tastes. In Massachusetts we have quite a number of small towns taking their water supplies from ponds and small reservoirs. Many of these supplies are

*New Rochelle Water Co., New Rochelle, N. Y. †Consulting Engineer, Boston, Mass.

chlorinated, and it often happens that the waters are rather highly colored with a varying organic content in the water due to the vegetation on the water shed.

It is natural to attempt to keep the chlorine down and to eliminate the chlorine taste as far as possible. The control of residual chlorine I think is not all that could be desired. The chlorination is oftentimes set to give a certain residual under certain conditions, and there it remains.

Mr. Enslow points out that, using the ammonia-chlorine process, the amount of chlorine can be increased materially without causing tastes, and I think that this would apply especially to the small supplies of this kind, where the operators are more inclined to govern their actions according to complaints from tastes than from the maintaining of the proper hygienic quality of the water. Take a sudden freshet in the fall, when the organic content of the water increases suddenly. They have been using, let us say, 1 p.p.m. of chlorine all summer, and the chances are pretty good that they will continue to use 1 p.p.m. during the heavier load, and the residual chlorine may be reduced very materially. I feel that the ammonia-chlorine process as applied to such supplies would be very valuable.

There is one other thing I want to say. Mr. Enslow pointed out what you might call a model treatment, and of course it might not apply in New England but it might apply in the Middle West, perhaps. That is the use of chlorine, ammonia, lime, and many other things. I am perhaps a little conservative, but my feeling is that as far as possible we should try to encourage the use of our natural waters. I do not believe that we know yet the physiological effects of all the chemicals we may add to water supplies. As many of you know, the Massachusetts Department of Public Health has always been inclined to favor slow sand filtration as against mechanical filtration, with the use of alum or coagulants. I feel in a way that perhaps that attitude is justified, because I do not see where the end is going to be in the matter of adding chemicals. I do not want to appear ultra conservative, but I do think that there is that point to be considered, that wherever possible we should consider the hygienic quality of the water first, and if it is a matter of filtration versus the addition of many chemicals, we should give serious consideration to filtration; in other words, we should come as close to getting the natural water supply as we possibly can.

MR. ENSLOW. I do not think anybody in the world can object to that attitude. We just naturally find, however, that where one is up against it temporarily, until such time as he can obtain the natural or God given purity, he has to do something about it.

BEST METHODS OF CUTTING LINED PIPE.

*Superintendents' Discussion, Lead by David A. Heffernan.**

[*September 21, 1933.*]

I am greatly interested in the subject "Best Methods of Cutting Lined Pipe" which was discussed at the American Water Works Convention in Chicago in June, 1933. Various speakers expressed their views in regard to making the cuts; the tools mentioned were the wheel cutter, dog chisel and the diamond point. Yet, I have come home from Chicago still in doubt as to what is the best method for cutting lined pipe, especially de Lavaud pipe. If I have remembered correctly, Mr. Newsom during his discussion seemed to favor the use of the dog chisel. From the following opinions, which were voiced at Bridgeport in September, there may be gathered some information which may be of value in the future when pipe cutting.

Several kinds of pipe were discussed. Let me quote the remark of Mr. Gibson,† who has been using cement lined pipe since 1922. "We have purchased standard pit-cast pipe, de Lavaud centrifugal cast pipe, and Monocast and Sandspun pipes, and have handled them all without increase of cost or breakage." In speaking of cement-lined pipe, Mr. Gibson explains, "At first we handled it with kid gloves so to speak, fearing that jarring or hammering would cause the cement lining to shell out and leave the pipe. Experience, however, has taught us that this extreme care is unnecessary. *Cutting:* "We have used the ordinary form of cold cut and sledge hammer for cutting the pipe, as well as the improved wheel type of cutters. We find either process to be perfectly safe in handling; we do find, however, that the wheel cutter tends to cause the cement to break out further back into the pipe from the cut edge than it does with the use of the ordinary cold cut cleaver, and sledge. Just exactly why is hard to satisfactorily explain."

In Milton we have been using cement lined mains for a period of eight or nine years with very good results. In cutting de Lavaud pipe we use the wheel cutter and on Sandspun pipe the dog chisel and sledge. Both of these methods do not materially disturb the lining of the pipe. Various tests were made in cutting of the de Lavaud pipe before our present method was adopted.

PERCY S. WILSON.‡ Do you refer to the straight edge chisel or the diamond point?

MR. HEFFERNAN. The straight edge chisel.

ROGER W. ESTY.§ I have used both kinds of pipe, sand spun and U. S. In fact, from information that I get this year, the U. S. concern claims that they have a pipe which is equal to the sand spun pipe, that you can

*Superintendent, Milton, Mass. †Commissioner of Public Works, Charleston, S. C.
‡Superintendent of Operation, Community Water Service Co., New York, N. Y.
§Superintendent, Danvers, Mass.

bang it and cut it, and do anything you want to with it and it will act just the same. We did have some sad experiences in the beginning, but with both types of pipe. I make all cuts with a wheel cutter, using double chain tongs, one going each way. My men would rather use the wheel cutter than to use the hammer and chisel. It makes a better cut and does not disturb the lining. It is practically a one-man job that way, and you do it faster.

PRESIDENT ELLIS. How large pipe do you cut?

MR. ESTY. We cut 12-in. pipe that way.

PRESIDENT ELLIS. What about pipes of larger diameter.

MR. ESTY. We have done the cutting with hammer and chisel up to 20-in. pipe.

PERCY R. SANDERS.* I have cut de Lavaud and sand-spun pipe up to 12 in. in diameter. We always use chain tongs and a wheel cutter. On the old style, pit-cast pipe, we used what Mr. Heffernan calls the dog cutter, or cold chisel. We find we have better luck cutting pipe with the wheel cutter — either kind of pipe.

W. R. CONARD.† If I were cutting pipe I would use neither the dog chisel nor the wheel cutter; I would use a machine cutter. I believe that this is the only logical equipment to use on any kind of pipe. You get a clean cut; you have a piece of pipe you can use over again if you take the section out. If you use a machine cutter there is no question as to injury, whether you are dealing with tar-coated, cement-lined or bitumastic-lined pipe.

MR. HEFFERNAN. Mr. Conard's remarks are very good. If there can be placed on the market a cutter that will not cost as much as $500.00 and one that can be carried around on the job without having to obtain different sizes and running up expenses, I will agree with him. If he can recommend some machine cutter at a low cost we will consider it.

MR. CONARD. There is a machine cutter that is produced in Ohio for 4, 6, and 8-inch pipe, which costs $75;

MR. HEFFERNAN. That is reasonable enough.

MR. CONARD. We have them and use them.

MR. ESTY. Is that cutter hand-operated?

MR. CONARD. Yes.

MR. ESTY. How long does it take to set up the machine?

MR. CONARD. Not more than five minutes.

MR. WILSON. I agree with Mr. Conard's remarks. I want to ask him if he does not think that a skilfully used diamond point is about equivalent to the cutter, because the skilfully used diamond point is something that can be applied without the first expense of the cutter, and if the men are skilful they can cut off the pipes so that the ends are nearly as true as when a machine cutter is used. At the same time, a diamond point does not deliver the shock to the pipe in such a way that it cuts out a piece of metal and does not tend to shatter the same as a straight edge.

*Superintendent, Concord, N. H. †Inspecting Engineer, Burlington, N. J.

MR. CONARD. That would depend upon whether you are cutting it on the bank or in the trench. If you are cutting the pipe on the bank you can with very good success use a diamond point to make your cut, but if you are working in the trench, there would be only one answer for me.

MR. WILSON. You can't use the diamond point in the trench unless you can revolve the pipe.

THE DAYS OF SMALL THINGS.

BY CHARLES W. SHERMAN.*

[*Received October 15, 1933.*]

A report of the Directors of the Cambridge, Mass., Water Works for the year 1860 is interesting particularly as showing what small sizes of pipes were considered adequate for a city of 26,000 persons. The works had been in operation only three or four years,† and included 124 517 ft. of pipe, as follows:

Diameter of Pipe—inches.	Length—feet.	Per cent. of Total
20 (intake)	180	0.1
12	5 950	4.8
10	13 180	10.6
8	6 000	4.8
6	14 955	12.0
4	56 263	45.2
3	27 989	22.5
Total	124 517	100.0

No description of the pump or reservoir is contained in the report, but city records furnished by Hon. T. W. Good, Superintendent of the Water Works, show that there were two Worthington pumps each of 1.5 m.g.d. capacity, and that the reservoir, which was 73 ft. higher than Fresh Pond, and was at the corner of Reservoir and Highland Streets in Cambridge, had a capacity of 1 764 000 gal. after being rebuilt and enlarged by the city in 1867. The total number of customers is not stated in the old report, although there was a gain of 575 during the year.‡ The estimated revenue from water rates for 1861 was $20 000.

"About the middle of July a second fireman was employed, and from that time to the present the pumps have been worked 24 hours per day... Besides the saving in wear and tear of the machinery, we find a saving of 2 400 pounds of coal and 145 pounds of wood per week, notwithstanding the quantity of water raised is increased. This saving is sufficient to meet a large portion of the expense of an extra fireman, for which we pay $32 per month."

The estimated operating expenses for 1861, namely all expenses except interest, dividends, and sinking fund charges, were $5 500, as follows:

*Of Metcalf and Eddy, Consulting Engineers, Boston, Mass.

†The corporation was organized in 1853.

‡This appears inconsistent with the total number of water takers (300) at the time the works were acquired by the city of Cambridge in 1865.

Salaries of officers	$2 000
Expense at the pond	3 000
Incidental expenses	500
	$5 500

During the first year of the city's ownership, 1865, the receipts were $32 367, and the expenditures $25 170. The salary of the superintendent was $625. The average daily water consumption was 975 000 gal.

Can there be any doubt that 1860–61 represented the days of small things, so far as water works were concerned?

RECENT ADDITIONS TO THE CAMBRIDGE WATER SUPPLY.

BY HOWARD M. TURNER.*

[*Read on November 8, 1933.*]

Existing Water Supply. The water supply of the city of Cambridge was taken originally from Fresh Pond in Cambridge in 1856. This supply was supplemented in 1887 with water brought in a conduit from a storage reservoir constructed on Stony Brook, a tributary of the Charles River, in Waltham and Weston. In 1897 larger storage reservoirs were completed on Hobbs Brook, a tributary of Stony Brook, in Waltham, Lincoln and Lexington. The water from the Hobbs Brook reservoirs flows in the channel of Hobbs Brook to Stony Brook Reservoir and is drawn through a conduit to Fresh Pond, whence it is pumped for the supply of the city. See Fig. 1.

Stony Brook reservoir contains 402 m.g. of water. The dam is of earth and is founded on the natural material, a large part of which is of a sandy and gravelly nature and there is reason for believing that considerable leakage occurs. The total drainage area is 23.75 sq. miles and the net drainage area below Hobbs Brook reservoir is 16.5 sq. miles. The reservoirs on Hobbs Brook are much larger than the Stony Brook reservoir, containing when full approximately 3 000 m.g. The watershed area directly tributary to the Hobbs Brook reservoirs is only 7.25 sq. miles.

From Stony Brook reservoir the water is carried to Cambridge by gravity. For the first 4 300 ft. the pipe was cast iron, 36-in. in diameter. Thence there were 8 700 ft. of 30-in. cast-iron pipe extended through the Mt. Feake Cemetery and along the Charles River between the railroad and the river, across Newton Street to the beginning of a concrete conduit. The concrete conduit is 26 700 ft. long, 63-in. in diameter, laid on the hydraulic grade line with the exception of three siphons, and extends to Fresh Pond where a short length of 40-in. steel pipe leads to the filtration plant and an overflow weir permits discharge into the pond. The capacity of 12 000 ft. of cast-iron pipe with Stony Brook Reservoir full was 14.7 m.g.d. and less with the reservoir drawn down. This was barely sufficient to serve the needs of Cambridge at the time. The 63-in. concrete conduit, on the other hand, has a capacity running full of about 33 m.g.d., and may therefore be considered to be ample in size for any possible future use.

Fresh Pond is a natural lake in the western end of the city with an area of 88 acres. It contains when full a total of 1450 m.g.; 400 m.g. of this were available by means of low-lift pumps in the purification works. Water thus taken from Fresh Pond could be replaced by excess water from Stony Brook and by the natural runoff of the Fresh Pond watershed, estimated to be about 1.25 sq. miles.

*Member of Cambridge Water Board, Cambridge, Mass. and Consulting Engineer, Boston, Mass.,

The purification works were built in 1922 on the shores of Fresh Pond. They are of the mechanical or rapid sand filter type. There is a receiving chamber into which water is introduced either directly from Stony Brook, or by means of the pumps from Fresh Pond. From the receiving chamber the water flows into the coagulation basin, a covered masonry structure of

FIG. 1.—PRESENT CAMBRIDGE WATER SUPPLY.

1.5 m.g. capacity, divided in the center for ease of cleaning. After subsidence in this basin the water is brought directly to the pipe gallery of the filter plant, which comprises 10 units of an area of 480 sq. ft. each, and having a total nominal maximum operating rate of 120 m.g.d. per acre or a total capacity of 14 m.g.d. From the filters the water flows through an aërator to a covered concrete pure-water reservoir located under the filter beds holding approximately 4 m.g.

The pumping station is located near the purification works and is supplied from the pure water reservoir. The pumping station contains a new 20-m.g.d. Worthington cross-compound steam pump put in three

years ago and three other older pumps with a total capacity of 30 m.g.d. The water is pumped up to the distribution reservoir in Payson Park holding 22 m.g., whence it goes to the distribution system through a 40-in. steel main.

In 1930 there were 16 800 services of which 9 000 were metered.

Proposed Enlargements. Late in 1929 the matter of additions to the water supply were considered. The consumption had grown to over 13 m.g.d. equivalent to about 115 g.p.d. per capita with something in excess of 50 per cent. of the services metered (Fig. 2*). Pressure in the eastern end of the city was not satisfactory.

Possible methods of increasing the water supply had been considered for many years. Consideration of the capacities of the Stony Brook and Hobbs Brook reservoirs shows that Stony Brook Reservoir with a capacity of 24 m.g. per sq. mile is too small to store all the water from its catchment area whereas the Hobbs Brook reservoirs with a capacity of 410 m.g. per sq. mile are too large, the result being a waste of water every year from Stony Brook Reservoir. In a report made in 1912 Frederick P. Stearns estimated the safe capacity of the system as it then existed at 11.5 m.g.d. Raising Stony Brook reservoir later increased this to 11.7 m.g.d. Fuller & Everett recently rated the existing capacity of the Stony Brook supply at 12.1 m.g.d. in a 95 per cent. dry year with an additional possible 0.9 m.g.d. from Fresh Pond, a total of 13 m.g.d. It was clear that with a growing consumption of over 13 m.g.d. some extension of the supply was necessary.

Definite steps were taken early in 1930 by the employment of Allen Hazen to give a preliminary opinion and in November, 1930, a complete study of the system was made by Mr. Everett of Hazen & Everett. At that time we were experiencing a low-water period which showed very clearly the necessity of further supply, the reservoirs being depleted so that there were only 50 days of water remaining in the system.

Driven Wells. Various plans were considered; among them driven wells. Test wells were put down near Fresh Pond where it was thought water might be obtained. An adequate supply of water which would not require filtration or carrying long distances in a pipe line presented an attractive possibility. Six test holes were drilled to bed rock at varying depths from 94 to 173 ft. The character of the material encountered was such that it was felt that there was no assurance that the underground supply was sufficient. The water found was extremely hard and even when mixed with the other water would have raised the hardness of the whole supply from 45–60 to over 60–90 p.p.m. depending on the proportion of water from the driven wells.

Use of Stony Brook Waste. The recovery of the water wasted at Stony Brook Reservoir, amounting on an average to about 2 700 m.g. a year (7.4 m.g.d.), has long been considered a possible additional supply. Mr.

*Fig. 2. shows actual quantities from 1890 to date, the predictions made in 1912 by L. M. Hastings, City Engineer, which compared very closely to those made at the same time by F. P. Stearns, Consulting Engineer, and also those made in 1930 by Fuller & Everett.

Stearns in his report of 1912 proposed pumping this excess water from Stony Brook back into the Hobbs Brook reservoirs. At various times it has been suggested that the level of the banks of Fresh Pond be raised so that additional water could be stored there. The plan now investigated was the

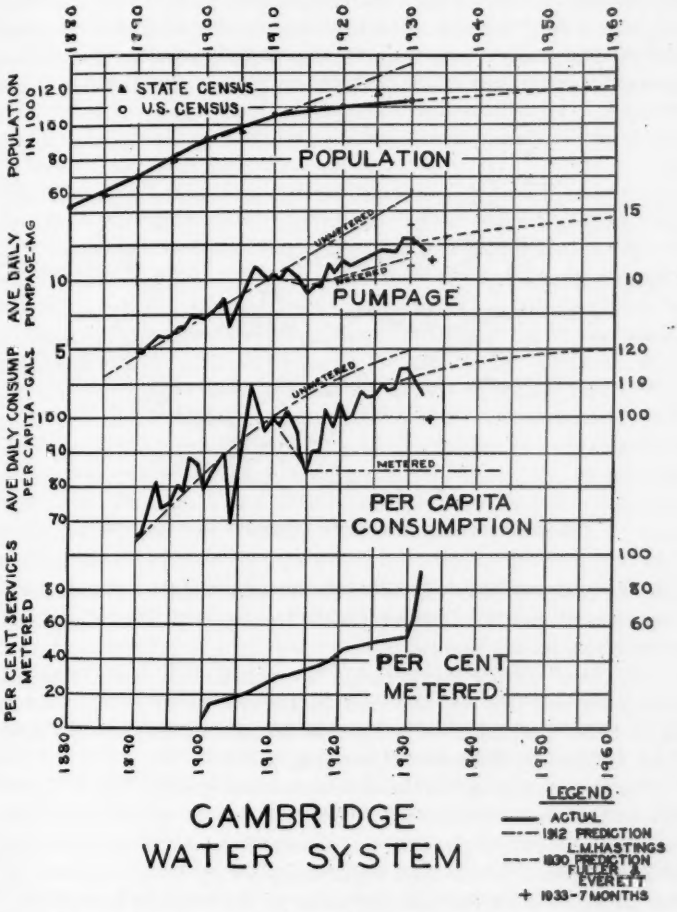


FIG. 2.—POPULATION, PUMPAGE, CONSUMPTION AND METERAGE OF CAMBRIDGE SUPPLY.

use of more of the existing capacity of the pond. Soundings showed that it is very deep, 55 ft. in the deepest place, and the additional storage to take care of the overflow from Stony Brook as it might be necessary could be stored if the water could be drawn low enough to make room for it. The

existing limit of draft, 9 ft., with the pumps available allowed a storage of 400 m.g. A draft of 30 ft. would make immediately available 800 m.g. additional. By increasing the size of the conduit leading from Stony Brook so that some of the flood water from that reservoir could be carried into Fresh Pond it was estimated that approximately 1.9 m.g.d. could be added to the present supply making a total dependable yield of about 14.9 m.g.d. throughout a very dry year. The total cost of this work was estimated at $275 000. The yearly charge for pumping would be small as it would not be necessary to pump very much except in dry years. The amount of pump-

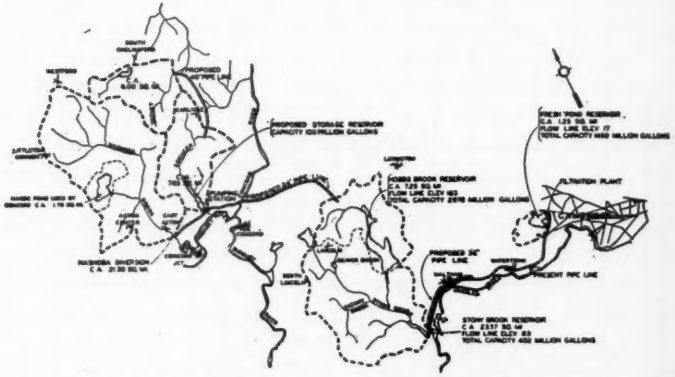


FIG. 3.—PROPOSED ADDITIONAL SUPPLIES FOR CAMBRIDGE.

ing in any ordinary year will be for about three months. This was clearly an economical source of water which developed to its full extent the present resources of Cambridge.

Additional Future Sources. An investigation of additional sources available beyond this was also made by Fuller & Everett. The most promising of these was found to be Spencer Brook and Tophet and Nashoba brooks, tributaries of Assabet River (Fig. 3).

The first step would be the development of Spencer Brook, the area of which at the proposed point of diversion is 7.5 sq. miles. The catchment area is fairly clean and there are no towns or large sewers except that of the Middlesex School. There is an opportunity for an economical storage reservoir in that section enabling the water of the brook to be stored so that the pumping plant and diversion pipes may be smaller. The development would be carried out in two steps as the water was needed. The first would be a small dam storing about 100 m.g. on an existing pond and a pumping station with a capacity of 8 m.g.d. with a 24-in. pipe about 30 000 ft. long to the Hobbs Brook reservoirs. This would add 4 m.g.d. to the capacity of the system in a 95 per cent. dry year. The cost of this work was estimated to be $600 000.

The second step would be increasing the size of the reservoir by raising the water level 26 ft. impounding approximately 2 billion gallons of water. Additional drainage area from Tophet Brook with 6 sq. miles of catchment area would be brought in by gravity into this reservoir. Additional pumps and a second pipe line to Hobbs Brook would be installed. This step would add an additional 5 m.g.d. making the total yield from the Spencer Brook watershed 9 m.g.d. The cost was estimated to be $1 120 000.

Nashoba Brook drainage area is directly west of that of Spencer Brook. The area above the point of proposed intake is 21.3 sq. miles. Should it be found necessary to supply more than the 9 m.g.d. additional to the present supply this area presents another possible source. An additional 3 m.g.d. could be obtained by this.

Entering the Metropolitan System. Another plan considered was entering the Metropolitan System. Cambridge did not enter the Metropolitan when it was formed in 1895. The subject has naturally come up from time to time and was raised in 1920 at the time the matter of filtration was being considered, but it was decided then to use Cambridge's own supply with filters. The Metropolitan District was approached in 1930 to see on what terms Cambridge could enter in order to provide for additional supply needed then and in the future. It was found that Cambridge would be required to pay an entrance fee of $2 620 000 plus a yearly assessment covering its share of the cost of maintaining and operating the works based on the ratio of its population and consumption to that of the other municipalities of the District. The Commission estimated that this assessment under the conditions then existing would amount to $312 000 a year. If Cambridge should operate its own works it would be allowed a rebate of $44 per m.g. on all such water produced. The $312 000 per year amounted to about $66 per m.g. so that the net cost to Cambridge for water produced from its own works would be $22 per m.g. To this would be added the fixed charges on $2 620 000. In 1930 the total expenses of running the Cambridge water works including fixed charges of interest, sinking fund and serial bond retirements was about $468 000 a year. Using Metropolitan water this cost would be reduced by an allowance of $44 on the 4 700 m.g. or $207 000 leaving a sum of $261 000. To this would be added the $312 000 yearly assessment from the Metropolitan and the interest and funding of $2 620 000 entrance fee amounting to, at 7½ per cent., $196 000, or a total of $769 000 a year, thus increasing the total yearly cost by $301 000 or an increase of 64 per cent. When this extra cost is considered from the point of view of the additional water required it is very high indeed. Clearly with this system of charges Cambridge could not possibly afford to join the Metropolitan System at present. Cambridge which had its own source capable of safely supplying about 12 m.g.d. and only needed a supply for its small increasing use would be required to pay a maintenance charge on its *whole* consumption 33 per cent. greater than the credit it was allowed and in addition an investment charge on its whole

requirement with no credit at all for its own investment in its own supply works.

In 1919 the matter of entering the Metropolitan System was also considered and at that time the entrance fee was $1 134 000. In 11 years this had grown to $2 620 000. With this system of charging it is unlikely that Cambridge can ever go in to the Metropolitan System. When the increased amount of water now obtained from the complete utilization of its own watershed is not sufficient it will certainly be advisable for it to seek its own independent supply rather than to pay an entrance fee which then apparently will be in excess of the present huge sum, and at the same time paying a yearly charge greater than the credit it receives for supplying water from its own system. The question was not really acute at this time because it was clear that the cheapest water to get was that which it now owned but wasted. The problem will become important when more water is needed.

The Joint Report on the Metropolitan Water Supply made in 1922 lists 16 cities and towns within the 10 mile radius which were not members but might be expected to join in the use of the new Ware and Swift river supplies. As far as I know none of them has yet come in. Like Cambridge many of these may not be able to afford to enter the Metropolitan System under the present methods of charging for water.

Plan Adopted. Comparison of these three different sources of supply, wells and utilization of the waste of Stony Brook reservoir with a future extension if required to Spencer and Nashoba Brooks, led to the decision to start the latter program at least as far as the utilization of the waste from Stony Brook. The total cost of the 2 m.g.d. including all fixed and operating charges for the increased supply and the additions at the purification works was estimated to be $71 000 a year or about $97 per m.g.

It was also decided to start on a program of metering which would complete the metering of the whole city within about 4 years, and it was expected that with the additional 2-m.g.d. supply and the slowing up of the growth of consumption by metering, additional sources of water would not be required for some years.

Additions to Purification Works. The increase in consumption of water required additional purification works. The original filters were 10 in number but the foundations were constructed to allow for 6 more. The engineers recommended the installation of these 6 filters and in addition the construction of a new sedimentation basin of 3 m.g. capacity.

The six new filtration units increase the maximum capacity of the plant from 14 m.g.d. to 22 m.g.d. suitable for an average output of about 18 m.g.d., a proper size for an addition at that time.

The engineers also recommended that the dirty wash water formerly returned to the coagulation basin with detrimental effects on the purification or wasted to the sewer be returned instead to Fresh Pond by a 12-in.

pipe to a point remote from the pump intakes thus saving it but giving opportunity for dilution and clarification before it reaches the intake.

Increases in Distribution System. The Payson Park distribution reservoir is located at the extreme westerly end of the system and is connected to the pumping station by a 40-in. steel main and to the heart of the distribution system by a second 40-in. main. This latter main passes near the pumping station and is cross-connected to it by a pair of pipes of 30-in. and 24-in. diameter.

From the termination of the 40-in. feeder from Payson Park Reservoir at the Cambridge Common, three major arteries run down through the city in an easterly direction, a 24-in. main on Cambridge St. as far as Ninth St., a 20-in. main on Broadway as far as Third St. and a 20-in. main from the Common to Mt. Auburn St. increasing to 24-in. on Mt. Auburn St. to Green St., where it again becomes a 20-in. main as far as State St. and runs as a 20-in. main on State St. to Main St. The main central thoroughfare through the city, Massachusetts Ave., is practically completely occupied by the subway so that it contains only a 12-in. pipe. There are also other arteries of smaller pipes thrown out to carry water to districts lying in the other parts of the city. These are for the most part of 12-in. pipes. Subsidiary piping in the various districts are of 8-in. and 6-in. pipe with some 4-in. pipe.

The 20-in. pipe on Broadway was very old, having been laid in 1867, and was in very bad condition from electrolysis. After consideration of several plans including (1) the use of elevated tanks in the eastern part of the city; (2) the provision of a new 30-in. steel feeder main from the 40-in. main feeder in Brattle St. to the lower end of the city, south of the subway; and (3) the replacement of the Broadway pipe with a larger 30-in. pipe, the latter was finally chosen as providing the most economical solution of the problem. In addition to this, two 24-in. pipes were carried across to the main feeder mains on the south side of the subway and various 12-in. pipes were added.

Cost and Finance. Early in 1930 appropriation was made of $800 000 to cover the cost of the work except the enlargement of the sedimentation basin which was postponed until a later date. It was financed by the use of $210 000 of the Water Department surplus and by the issue of $590 000 of 3½ per cent. 25-year serial bonds. The amount from surplus was used to pay for the pumps at Fresh Pond, the new filter beds, the 12-in. mains and the meters, items which if paid for by bonds would require retirement in less than 25 years.

Fresh Pond Pumping Station. Figures 4 and 5 show the plans of the pumps and intake at Fresh Pond. Two pumps of 7.5 m.g.d. capacity each are located in a pump chamber at the shore of the pond and two pipes, one terminating 22 ft. and the other 32 ft. below high-water level are carried out into the pond down the slope. These steel pipes are laid for the most part in a trench dredged in the bottom of the pond except in some cases

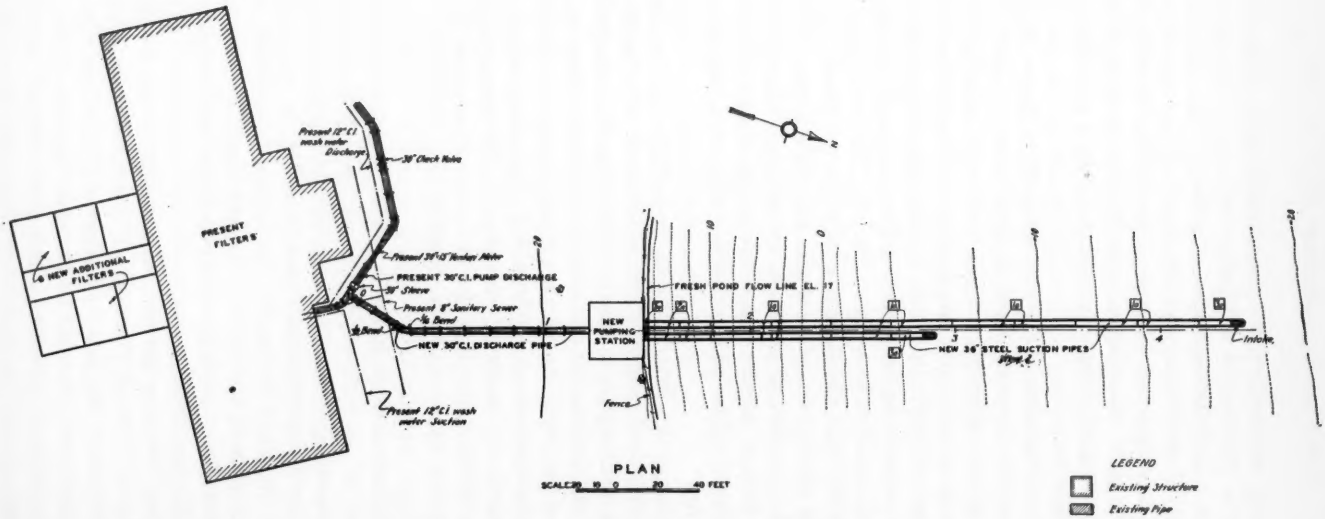


FIG. 4a.—PLAN OF NEW INTAKES ON FRESH POND.

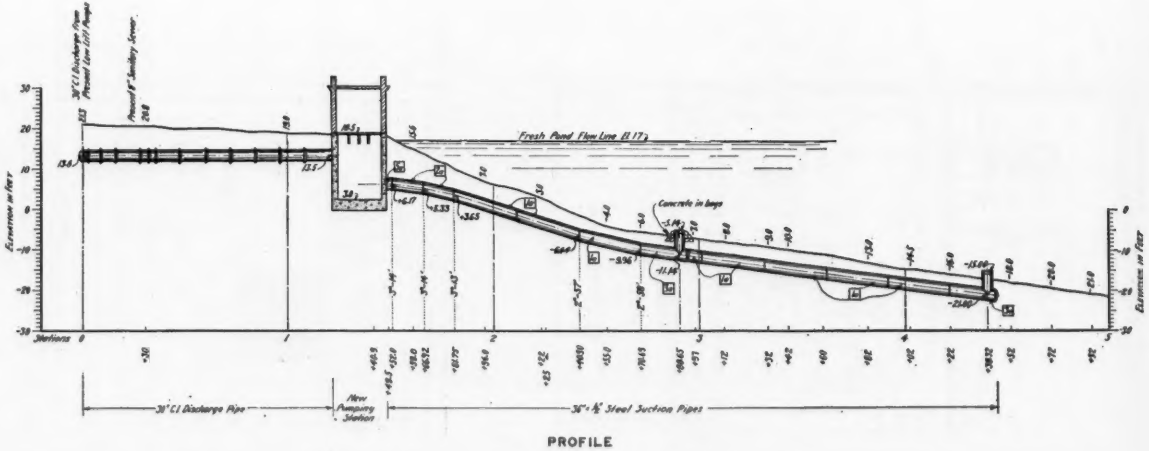


FIG. 4b.—PROFILE OF NEW INTAKES ON FRESH POND.

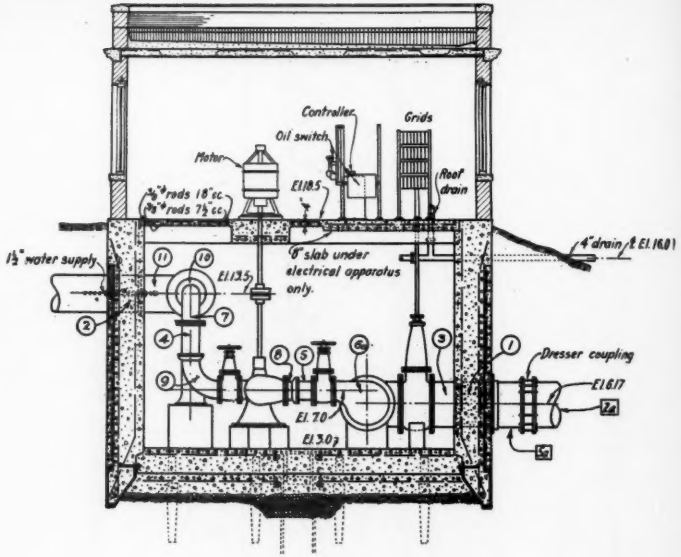


FIG. 5a.—SECTIONAL ELEVATION OF NEW PUMPING STATION ON FRESH POND.

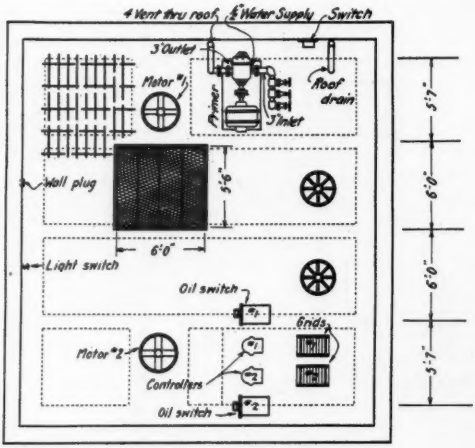


FIG. 5b.—FLOOR PLAN OF NEW PUMPING STATION ON FRESH POND.

where a soft bottom required pile cradles. The pipes are electric-welded steel pipes made by the Walsh Steam Boiler Works. They were assembled on pile trestles and lowered into place. The joints are Dresser couplings which were finally tightened up by a diver after installation. After testing the pipes for tightness the trench was backfilled. The pump house itself was excavated in the open using wooden sheeting, the contract being let to John M. Tobin & Co., of Boston. The whole bottom was waterproofed.

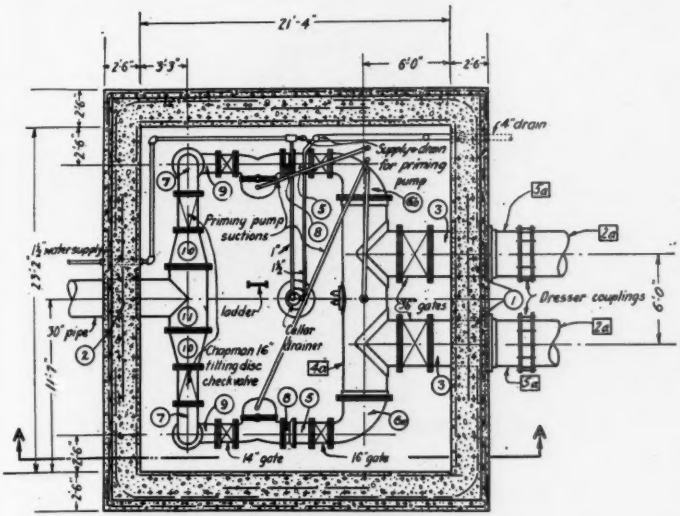


FIG 5c.—BASEMENT FLOOR PLAN OF NEW PUMPING STATION ON FRESH POND.

A sump in the middle provides for pumping out any possible seepage or condensation. The pumping equipment was furnished by the Turbine Equipment Co. Fuller & Everett were the engineers.

Waltham Pipe Line. For the new 36-in. pipe line from Stony Brook, two alternate arrangements were considered. The first provided for a cast-iron pipe paralleling the existing pipe line for its full length. The present pipe passes through Mt. Feake Cemetery in Waltham following a narrow driveway bordered by cemetery lots with very little room for the laying of another pipe so that for some of the distance it would be necessary to substitute for the present pipe a single new 48-in. pipe. The other plan (Fig. 7) avoided this congested construction by means of a tunnel driven through rock underlying a hill in the cemetery. This shortened up the line by about 300 ft. This tunnel was about 1 800 ft. long and about 6 ft. in diameter

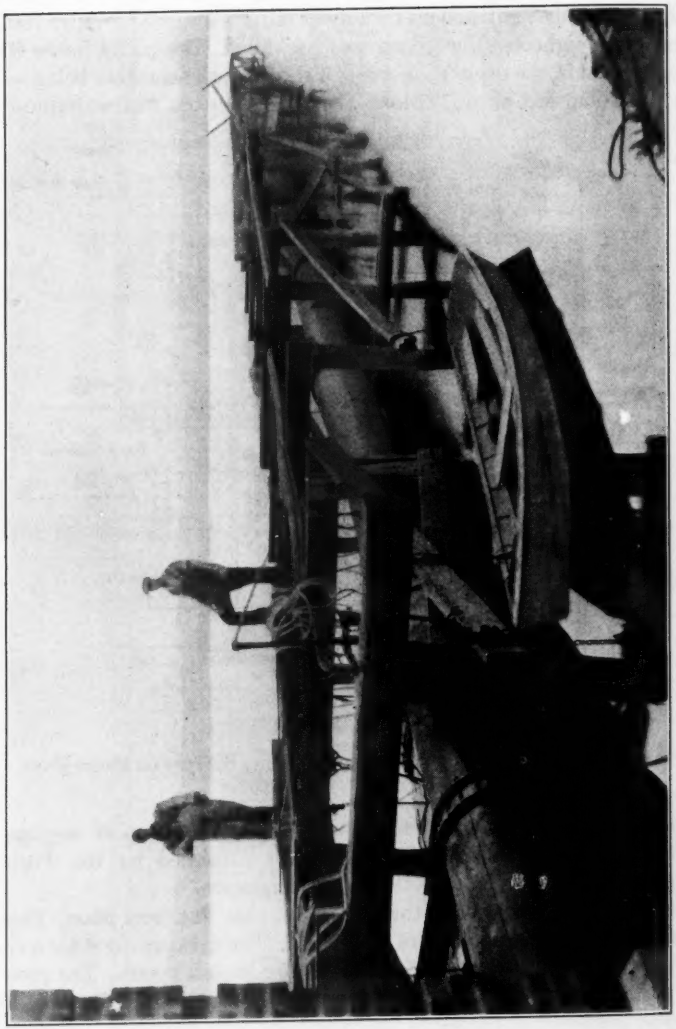


FIG. 6.—INTAKE PIPES ASSEMBLED PRIOR TO BEING LOWERED INTO PLACE.

with a 36-in. steel pipe in it. Bids were obtained on both plans. The tunnel route was estimated to cost more than the other route by about $11 000 after an allowance for rebuilding the roadways which would be disturbed on the other route. The fear of possible delay and damages to the properties in the cemetery led the Board to adopt the tunnel route even at the higher expense, this route being also favored by the cemetery authorities in Waltham. The contract for laying the pipe was let to John J. Coughlan & Co. The cast-iron pipe which extends from the lower end of the tunnel to the end of the line was furnished by the Warren Foundry Co. and the steel

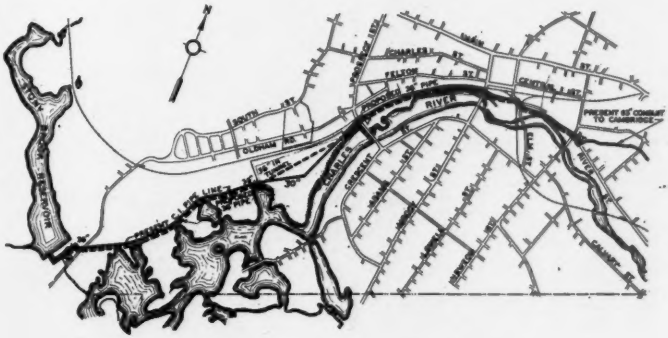


FIG. 7.—NEW PIPE LINE THROUGH WALTHAM.

pipe by the Walsh Holyoke Steam Boiler Works. The engineering on this work was done by the city engineer Mr. Hastings, and after his retirement by Mr. Hooker, with Fuller & Everett as consulting engineers.

There were many special problems in connection with the laying of the large cast-iron pipe through the streets of Waltham, under bridges, through mill yards and under the floor of a barber shop and the heating plant of a hotel, but the most interesting was that of the tunnel. It was not proposed originally to line the tunnel except where it was not in ledge but the character of the rock encountered in part of it made it necessary to line 1 050 ft. at considerable expense. Figure 8 shows the expansion joints, anchors and braces employed to prevent accumulated water from floating the pipe when empty as the invert is at times below the water level in the Charles River nearby. Ordinary tunnelling methods were used in excavation, all the work being done from the west end, the material being disposed of close to the tunnel portal in a low place which the cemetery authorities wished to have filled. The lining was placed from the west end by the pneumatic process. After the tunnel was completed the pipe was put in, three 30 ft. lengths being riveted together outside and then hauled into place on small trucks on the industrial railroad track.

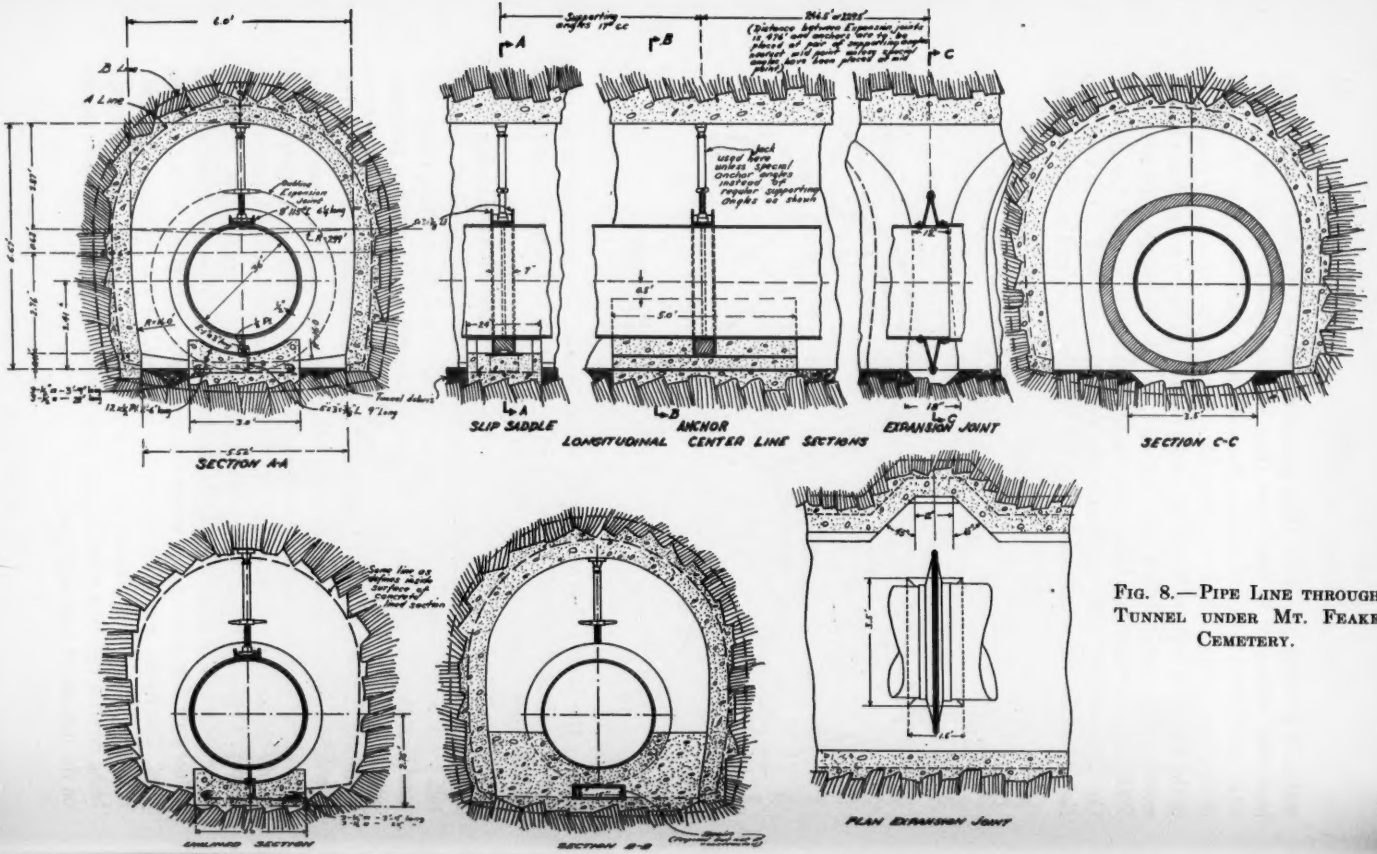


FIG. 8.—PIPE LINE THROUGH TUNNEL UNDER MT. FEAKE CEMETERY.

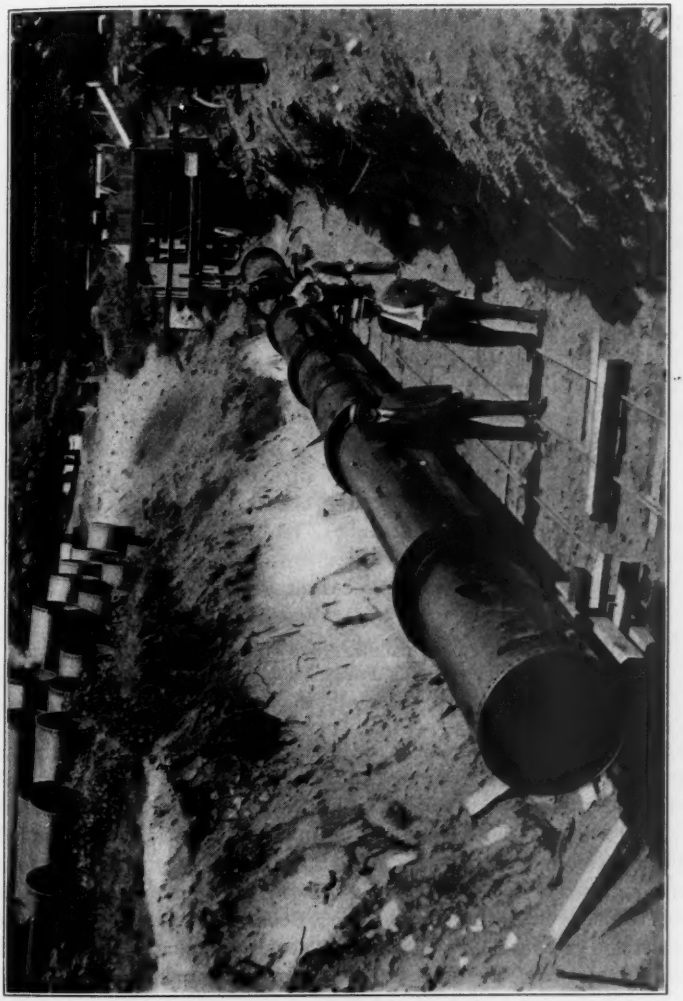


FIG. 9.—PORTION OF STEEL PIPE LINE BEFORE BEING MOVED INTO PLACE IN TUNNEL.

Filtration Plant. The work at the filtration plant (Fig. 10) consisted of the addition of six new filter beds, foundations for which were provided when the original plant was built. The filters have Wheeler bottoms the same as the original 10 beds. The piping is laid out with particular attention to the accessibility of the pipes and fittings. The wall of the building between the new and old filters was removed so that the new filters are in the same room as the old. The new wash-water disposal pipe was put in by

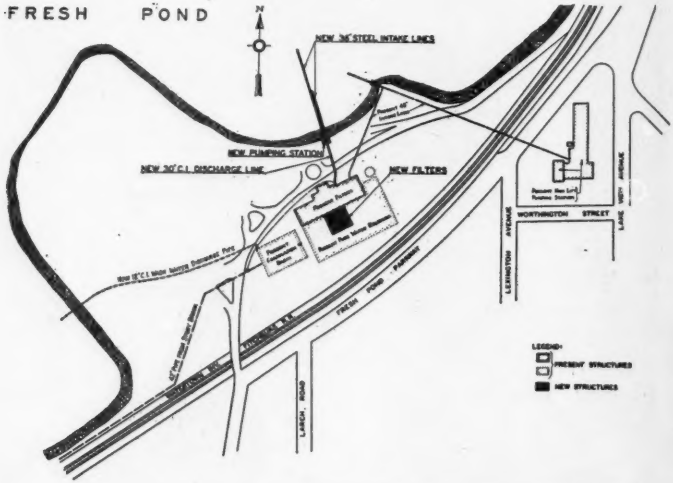


FIG. 10.—GENERAL PLAN OF ADDITIONS TO FILTRATION AND INTAKE WORKS ON FRESH POND.

the department's own men. The contract for the concrete work in the building was let to John M. Tobin & Co. The filter equipment and piping were furnished by the Roberts Filter Co. Fuller & Everett were the engineers.

New Mains. The new large mains in the distribution system were laid out by the city engineer's office. The Superintendent of the Water Board, Mr. Good, was in direct charge of the work. The laying of as large pipes as these through city streets with sewers, conduits and the various interconnections and services presented the usual problems. The pipe was laid along the south side of the street, generally, just outside the sidewalk line. The tees for the inter-connections at street intersections were included, and the inter-connections on the south side made as the work went along, the services being carried over the new pipe with copper pipes. The pipe was completed in sections after connection of the services and inter-connections on the south side. After the pipe was completely filled the old pipe was discontinued in sections and the services and inter-connections on the north

side connected into the new pipe. The latter work was done by the department's own men. The pipe was furnished by the Warren Co., and John M. Williams was the contractor for laying it. The restoring of the pavement was done by the street department of the city. In most cases a reinforced concrete sub-base was laid across the top of the trench. A large part of the pavement was not completed until some time after the pipe was laid so there was ample opportunity for settlement.

The work included the inter-connection of a 24-in. pipe on Trowbridge St. and one across the lower end of the city. All the 12-in. pipe was laid by the department's own men under the superintendent's charge.

Cost. The work was completed with the exception of some of the interconnections on Broadway late in the year 1931 and the final figures of total cost came within $20 000 of the estimate of $800 000, the additional appropriation being required for the tunnel work chiefly on account of the extra lining needed.

The pumpage during 1931 averaged less than 12 m.g.d. and so far in 1933 11.5 m.g.d. How much of this reduction in capacity from the high point of 13 m.g.d. in 1929 and 1930 is due to meters and how much to the depression it is difficult to say. There isn't any question that the latter is a considerable factor. On the other hand the fairly complete metering now in effect has not yet shown its full effects. The present capacity of the works, about 15 m.g.d., should be sufficient to provide for a good many years in the future. The land in Cambridge is pretty well built up. We look forward hopefully, however, for a continued industrial expansion which would mean an increase in water consumption but probably not a very great population increase.

When more water is required the question of whether Cambridge will obtain its own additional supply from another area which exists within easy reach or whether it can obtain water from the Metropolitan Supply on terms that will make the cost at all comparable will again arise.

Discussion.

Henry F. Beal.* As City Engineer of Waltham, the city through which the pipe line was constructed, I feel that I should say a word about the work. All construction was done in a workmanlike manner, every necessary replacement was made at no cost to the City of Waltham, and when the work was completed, except for a little temporary inconvenience perhaps, we should never have known that anything had happened. The handling of the work in such an efficient manner is due entirely to our good member, Timothy W. Good. We had no trouble from start to finish.

Lewis M. Hastings.† I am somewhat interested in this extension of the water works, as some of you may know. I would like to ask (1) how

*City Engineer, Waltham, Mass. †City Engineer, Cambridge, Mass.

much water can be obtained from the areas lying to the west of Hobbs Brook; (2) whether in the Broadway supply line any increase in head from the enlarged supply has been noticed; (3) whether in the tunnel section of the supply main from Stony Brook Reservoir any difficulty has been experienced in the maintenance of the tunnel.

TIMOTHY W. GOOD.* In answer to the first question, the first step of development at Spencer Brook is estimated to yield 4 m.g.d., the next step 5 m.g.d. additional, and the last 9 m.g.d. additional. In answer to the second question, the pressures along the Broadway pipe line have been raised from 7 to 11 lb. per sq. in., and that increase in pressure has extended in both directions, to the north as far as Cambridge St. As regards the third question, the water in the Charles River has been somewhat high. Although the City Engineer, Mr. Hooker, endeavored to make an inspection of the pipe, the water was such that he could not get into the tunnel to do it without pumping it out. We hope that when the water gets lower we shall be able to make an inspection. It has been in constant operation.

*Superintendent of Water Works, Cambridge, Mass.

CORROSION OF BRASS IN WATER SUBJECTED TO pH CORRECTION.

EDWARD W. MOORE.*

[*Read November 8, 1933.*]

In recent years there has been a tendency to minimize corrosion of water supply systems, in which the ferrous metals are predominant, by increasing the pH of the water. The possible effects of this procedure on the non-ferrous parts of the system have been questioned by Pirnie in his statement at the Pittsburg meeting of the A. W. W. A., that while corrosion might be reduced by increasing pH, zinc would be dissolved by too high pH. Nor would zinc be alone in such behavior. Any metal forming soluble complex salts would be susceptible to attack. Among such salts, in addition to the zincates, may be mentioned the aluminates, plumbates, and stannates. Thus zinc, aluminum, lead and tin might be attacked by waters, of high pH. Of these, zinc is the most important, as it is included in water supply systems in the form of galvanized pipe, and also as a constituent of brass pipe.

In order to determine what effect alkaline waters might have on the zinc in brass, and to study the general effect of pH on the corrosion of brass, a series of experiments on brass pipe (60 per cent. copper and 40 per cent. zinc) in water of pH 6.0 to 11.0 was carried on in the Laboratory of Sanitary Engineering at Harvard. Uniform sections of the pipe with surface of 20 sq. in. were placed in jars with ground glass covers, containing Cambridge water (normal pH 8.0) adjusted with hydrochloric acid or sodium carbonate to the following pH values: 6.0, 7.0, 8.0, 8.5, 9.0, 9.5, 10.0, 10.5, 11.0. Two waters of each pH were made up. In the higher pH range, this resulted in precipitation of calcium carbonate, which was permitted to take place before inserting the pipe. This allowed the solution to have free access to the pipe. One set was allowed to stand 60 days, the other 120 days, in a room maintained at a constant temperature of 25° C. All jars were shaken daily to reaërate the water and ensure uniform oxygen concentration.

It was already known that these solutions, depending only on carbonate equilibria for buffering, would not maintain constant pH. Therefore, periodic testing and readjustment with acid or carbonate was carried on. By this method the pH values were maintained within 0.2 of a unit. This is a tedious and time-consuming method, but no other seems to be available in experiments of this type. Sealed tubes, containing the same mixtures, do not maintain the original pH, but seem to be affected by the course of the corrosion reactions. Other and more stable buffer systems,

*Instructor in Sanitary Chemistry, Harvard Engineering School, Cambridge, Mass.

such as phosphate mixtures, would complicate results by the introduction of precipitates and coatings which would not be found in natural waters.

After the allotted time, the pieces of pipe were removed, and brushed into the solutions, so that any coating might also appear in the analysis. The solutions were then acidified. The method used for analysis of the solutions was a modification of that designed by Fairhall and Richardson[1] for zinc in biological material. It consisted essentially of preliminary precipitation of copper and zinc as sulfides, re-solution of the precipitate and separation of the copper as sulphide, the zinc remaining in the solution. The copper sulphide was then dissolved and the copper determined colorimetrically with potassium sulphide, as in Standard Methods. The zinc was determined nephelometrically as the ferrocyanide. This method proved very satisfactory. It was at least equal to electrolysis for copper, and very good for zinc. The determination of zinc in small quantities is exceptionally difficult, as it forms no colored salts. Experiments with known solutions indicated that the probable error is not over 10 per cent. Duplicate determinations were made. The results were corrected for the amounts of metals removed in the periodic pH tests. The correction was in the neighborhood of 10 per cent. or less of the amount found. The initial volume was 1 500 cc., the final 1 300 cc. for the 60 day tests, and 1 200 cc. for the 120 day. Results are shown in the accompanying table.

Perhaps the most striking result is the tremendous variation in the amount of zinc dissolved. In the case of the 120-day exposure, for example, we find 215 m.g. at pH 6.0 and 1.4 m.g. at 10.0. A definite minimum with respect to solution of zinc is shown to exist at pH 10.0. A minimum point is to be expected with a metal of this type, which is readily attacked by both acids and alkalies. This point coincides approximately with the point of maximum solution of copper. The amount of copper dissolved, however, is small in any case.

If the results are plotted logarithmically, as in the accompanying figure, that is, logarithm of the amount dissolved against pH, a clear picture of the behavior of the two metals is obtained. The logarithm of the amount of zinc dissolved declines roughly as a straight line with increase of pH, to pH 9.5. At this point, the rate of decline diminishes, and a minimum is reached at 10.0. Beyond 10.0 there is a slow rise in amount dissolved. The amount of copper dissolved drops from 6.0 to a minimum at 7.0, then rises to a maximum at 9.5 to 10.0. Beyond this point it falls slightly. With the exception of pH 6.0, the amount of copper dissolved seems to be inversely related to the amount of zinc, that is, as less zinc is dissolved more copper is dissolved and *vice versa*. This phenomenon may be readily explained by the electrochemical theory of corrosion, which may be expressed by the following equations:

$$Zn + 2H^{+} = Zn^{++} = 2H \text{ (atomic)}$$

[1] Fairhall, L. T. and Richardson, J. R., The Nephelometric Analysis of Zinc. Jour.Am. Chemical Soc. 52, 938 (1930).

The atomic hydrogen must then be discharged by one of two methods:

1. by dissolved oxygen: $2H + \frac{1}{2} O_2 = H_2 O$
2. by evolution as molecular hydrogen: $2H = H_2$

For a more complete account of the theory, the reader is referred to Speller, Corrosion Causes and Prevention (1926), p. 21 and following.

Speller (pp. 29–32) shows that when two metals are in contact, the hydrogen film is formed by the solution of the more anodic element, and the solution of the cathodic element is prevented, as would be expected from the relative position of the two metals in the electrochemical series. In the case of brass, zinc is the anodic element, copper the cathodic. The solution of zinc protects the copper from attack. The protective effect on the copper would diminish as the solution of zinc diminished, and thus somewhat more copper would be dissolved. In the case of pH 6.0, it may be that the limit of this protective effect has already been reached, and the greater amount of copper dissolved at 6.0 than at 7.0 merely reflects the increase in hydrogen-ion concentration.

A study of results at the same pH with respect to time may be expected to give some information on protective action other than that just discussed, such as the formation of films of the hydroxides or carbonates of the dissolved metals. If solution after 120 days is less than twice that of 60 days, we may infer that some sort of protective layer is exerting an appreciable influence on the rate of corrosion. The ratio of the amount dissolved in 120 days to that at 60 days will be found in the table. In the case of zinc we can by inspection of the ratios roughly delimit three separate zones, one from pH 6.0 to 7.5 or 8.0, in which such coatings are of slight or no influence because of too great acidity, one from 7.5 or 8.0 to about 9.5, where they exert a generally appreciable influence, and one from 9.5 on, where they are again of slight influence. In this latter zone, according to our theory, coatings of zinc hydroxide or carbonate would be dissolved to a greater or less extent as zincates by the highly alkaline medium. More data would be required to delimit these zones exactly.

In the case of copper, protective coatings generally seem to exert more influence the higher the pH. This might be expected, as copper does not readily form soluble compounds analogous to the zincates, even in quite alkaline solutions. Thus there is no tendency to dissolve any hydroxide or carbonate film at higher pH values. In very alkaline solutions, however, this might not be true. The very high ratio for pH 8.0 is probably due to error.

The practical implications of the experiment may be disposed of as follows. For brass piping, at pH values up to 10.0, the higher the pH, in general, the better the service to be expected, as dezincification will take place less readily at higher pH values. Possibly over long periods, pH values from 8.0 to 9.0 may give as good results as pH 10.0, because of the formation of protective films. Solution of copper from brass seems to be negligible from a service standpoint throughout the pH range studied.

Of course, all deductions are purely relative, and do not imply that brass would be essentially unsatisfactory at lower or higher pH values than the ones cited as best.

Summary. The results may be summarized as follows:

1. Dezincification of brass appears to decline from pH 6.0 to a minimum at pH 10.0, and then to rise slightly to pH 11.0.

2. Solution of copper from brass appears to depend on the amount of zinc dissolved, minimum zinc dissolved coinciding with maximum copper dissolved.

3. Protective films retard dezincification only in a limited range — about pH 7.5 to 9.5.

This experiment might be taken as a confirmation of the theory of the existence of minimum corrosion points for amphoteric metals such as zinc. It seems to indicate that the investigation and location of such points for pure zinc, or zinc coatings on other metals, and for the other amphoteric metals might yield results of practical interest. Similar studies on copper and other brasses might also be of value.

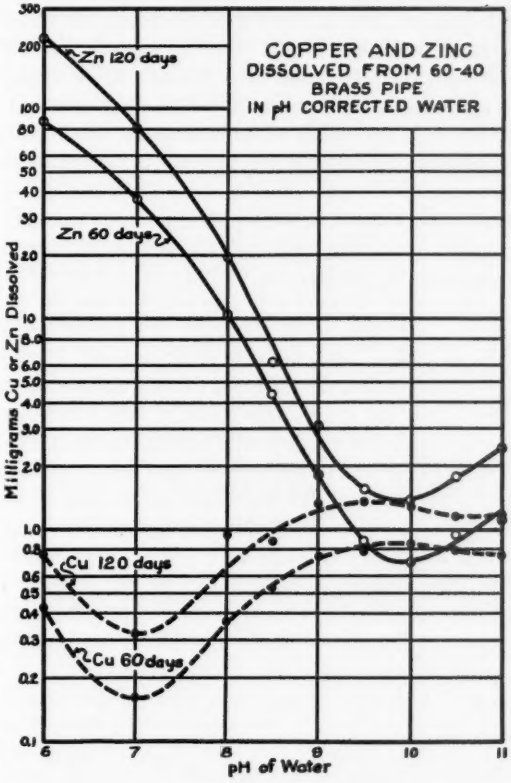
RELATIVE AMOUNTS OF COPPER AND ZINC DISSOLVED FROM 60–40 BRASS IN pH — CORRECTED WATER.
Exposed Surface Area 20 sq. in.

pH.	COPPER — m.g.			ZINC — m.g.		
	60 days.	120 days.	Ratio of (3) : (2)	60 days.	120 days.	Ratio of (6) : (5)
(1)	(2)	(3)	(4)	(5)	(6)	(7)
6.0	.43	.78	1.8	86.4	217.4	2.5
7.0	.16	.32	2.0	37.3	82.2	2.2
8.0	.37	.94	2.5	10.7	19.5	1.8
8.5	.53	.89	1.7	4.4	6.3	1.4
9.0	.75	1.34	1.8	1.8	3.1	1.7
9.5	.80	1.35	1.7	.88	1.56	1.8
10.0	.85	1.28	1.5	.69	1.38	2.0
10.5	.80	1.13	1.4	.94	1.77	1.9
11.0	.75	1.12	1.5	1.10	2.40	2.2

An excellent paper on the corrosion of commercially pure zinc in waters of different pH has recently been published by Boetheli, Cox and Littreal.[1] A comparison of the data given in this paper with their results is interesting. As plotted in their paper, their figures for corrosion of zinc yield a curve of much the same shape as the author's curve for zinc dissolved from brass. Their minimum corrosion zone is flatter, extending from pH 10.0 to 12.0. They also assign a wide range of stability to hydroxide

[1] Boetheli, Cox, and Littreal — Effect of pH on Corrosion Rate of Zinc in Oxygenated Solutions. Metals and Alloys 3,73 (1932).

films, pH 6.0 to 12.0. These, however, are minor differences. They may be due to the different nature of the materials tested, or to differences in experimental technique. Boetheli, Cox, and Littreal removed CO_2 from their solutions by aëration, and their experiments were much shorter than those reported in this paper. The flat minimum corrosion range shown in their results might possibly be explained by the fact that the amount of corrosion



was determined by weighing the test piece. This procedure does not lend itself to the determination of the small differences (one milligram or less) which the author finds in the range 10.0 to 11.0.[1]

No matter how the minor differences between the two curves are explained, it is apparent from their general shape that the solution of zinc from 60–40 brass is affected by pH in much the same way as its solution from commercially pure zinc. The solution of zinc from a galvanized surface may be reasonably expected to show a similar variation with pH. To check this conclusion, experiments with galvanized pipe will be made subsequently.

[1] See Cournot and Chaussain. Comptes Rendus 194,1823 (1932).

Discussion.

F. G. Smith.* There are some conclusions in this paper which, according to the experience of the American Brass Company during the last ten years, may not always actually work out in practice. We know that in some localities where certain types of water are treated with lime to reduce corrosiveness and the pH is held at about eight, the treated waters are not very corrosive to 60–40 brass. But in other localities where the water is different, the same treatment resulting in a pH value, of say 8.5 or 9, will not reduce the corrosiveness upon 60–40 brass. Mr. Moore's paper would lead one to believe that a pH value of nine to ten in domestic water would give protection for 60–40 pipe but, according to our experience, this is not always true.

We have learned that mineral constituents in the water have as much bearing on corrosion as the pH value because those waters which are not very corrosive to yellow brass of the 60–40 type not only have a pH value of about seven, but they also carry enough calcium or magnesium bicarbonate to keep the water neutral and perhaps in themselves afford protection. On the other hand, if a very soft acid water is treated by adding lime to the point where phenophthalein alkalinity is noticeable, that water becomes corrosive to yellow brass because it is alkaline. Where there is no carbon dioxide in the water, no phenolphthalein alkalinity, and the temporary hardness is not too low, in other words, where the water is neither acid with carbon dioxide nor alkaline with normal carbonates, that water is not corrosive and the pH will be somewhere near seven but not nine or ten.

The other types of brass like red brass containing 85 per cent. copper, or pure copper, are not much affected by waters which are either acid with carbon dioxide or alkaline with normal carbonates.

I think that the conclusions reached by Mr. Moore lead one to believe a pH value of nine or ten is best for water where 60–40 brass pipe is used. According to our experience, this would not always be so, and one would be lost without knowing what the other characteristics are.

Mr. Moore. The concentrations of mineral constituents in waters certainly affect corrosion, possibly even more than the pH of the waters. That is why no anions or cations except those normally observed in all waters were added in adjusting the pH.

Mr. Smith. I know of certain cases where waters that are soft and acid, for example waters from swampy land very low in carbon dioxide but acid and corrosive, if treated with lime to the point where the pH value is nine still remain corrosive. Changing the pH value is not the whole remedy. Other factors must be considered along with the pH. I only know what those factors are through ten or twelve years' experience in handling brass pipe problems from all parts of the country and correlating corrosion data with water analyses.

E. L. Bean.† I think that Mr. Smith has hit on a point which was per-

*American Brass Co., Waterbury, Conn. †Chemist, Water Purification Works, Providence, R. I.

haps overlooked in the original paper, namely, the effect of concentration of anions and cations on solution, rather than the simple question of hydroxyl concentration. The concentration of these other metals will depress the point of the minimum solution of zinc in actual practice. I know from experience that, in a water which tallies somewhat with those described by Mr. Smith, an increase of the pH above eight, approximately, does increase the amount of solution of the zinc.

Possibly another thing besides the concentration of the metals is the concentration of the organic materials in the water, which has a tendency to protect zinc coatings.

PETER P. KOSTING.* In connection with the corrosion of brass, it has been explained that the brass goes into solution. Then you have no longer brass but copper and zinc ions. Something happens. There is a certain type of corrosion which leads to reprecipitation of the copper, which will explain the fact that at the lower pH values the copper content was found to be very low, whereas the zinc content of the water was found to be high. I was going to suggest a microscopic examination of the surfaces, but I don't know whether 60 or 120 days would be sufficient time to allow corrosion to become visible. The point I want to raise is, analyzing the solution for its metallic content after the copper has had the opportunity to reprecipitate, and in cases of high pH where the zinc hydroxide and copper hydroxide have had an opportunity to be precipitated, to judge the corrosion by the amount of zinc in the solution is not practicable.

MR. MOORE. The surface of the pieces exposed to pH 6 and 7 showed a color more like that of copper than of brass. This might indicate either a redeposition of copper or merely solution of zinc, leaving copper behind. If any zinc or copper hydroxide was precipitated on the metal, it was probably returned to the solution by vigorous brushing of the pieces as they were removed. That in the solution itself was, of course, dissolved when the solutions were acidified for analysis.

DAVID A. HEFFERNAN.† The subject, Corrosion of Brass Pipe, has been discussed in this Association for a great many years. The knowledge I have obtained at these meetings and from my own practical experience and observations makes me realize that soil corrosion has to be considered in the discussion of this troublesome problem. Soil corrosion of underground pipes is caused by chemicals in the soil and ground water. It cannot take place in the absence of moisture. Temperature, soil characteristics, conductivity and stray electric currents all affect the rate of corrosion. Electrolysis may occur between any two pipes laid in moist earth if these pipes include two or more different metals, or if there is other potential or electric conductivity. The grounding of electrical currents on water pipes and the possibility of damage through the increase in voltage, careless workmanship and inspection also add to the danger of corrosion.

I should like to know if there is a demand for 60–40 brass pipe now.

*Watertown Arsenal, Watertown, Mass. †Superintendent, Milton, Mass.

My reason for this question is that water-works men feel that by increasing the copper content to say 85–15 it would probably reduce the disintegration of the metal.

We all appreciate the efforts of Mr. Moore to correct the evil of corrosion of brass in water subjected to pH correction, but I doubt very much if he can get the desired results from laboratory tests because I believe other elements besides the water and material are factors in corrosion of brass. On continuing his tests I hope he will use 85–15 pipe.

MR. SMITH. Regardless of how much 60–40 brass pipe may be made I do not believe that many water-works superintendents use it for services. It certainly is not recommended for that use by the American Brass Company. In fact, the American Brass Company does not make this grade of pipe containing 60 per cent. copper, 40 per cent. zinc. Furthermore, we have learned from experience that yellow brass pipe is not suitable for water services as it may fail underground principally on account of stresses due to settling of the soil. For the past nine years we have been recommending and selling red brass pipe containing 85 per cent. copper, which Mr. Heffernan spoke of, and it is very satisfactory. At the present time, more pure copper is used for services than red brass.

Digressing a moment from the subject of the paper I will reply to Mr. Heffernan to say a word about electrolysis or galvanic action. So far as the experience of the American Brass Company is concerned, the small differences in the composition between the pipe and the fittings do not have very much effect on corrosion. Oftentimes pipes will leak at joints and it will be found that the fittings are made of high copper alloy and the pipe of a low copper alloy and then someone says "Electrolysis." As a matter of fact, if the fittings had been made of the same kind of metal as the pipe, the joint would probably have looked just the same. If you are a maker of brass pipe and sell it everywhere, you hear these stories. After many investigations you finally come to the conclusion that about half of what you hear is right and the other half you or no one else can explain satisfactorily.

Usually when pipes leak at threaded joints and electrolysis or galvanic action is blamed for the difficulty, the real trouble is due to the workmanship with which the line was put together. For example, we know in one location pipe of a certain kind carelessly assembled is in trouble in a relatively short time, whereas another installation by a good plumber is satisfactory after eight or ten years. The difference in the performance is not in the pipe or combination of pipe and fittings. It is due to the way the workman assembled the pipe. Such troubles crop up and electrolysis is blamed. I do not believe that electrolysis has very much effect. Theoretically, it has an effect but practically it is not so much as a great many think; at least our experience leads to that conclusion.

GORDON M. FAIR.* Mr. Moore's purpose, apparently, was to determine whether raising the pH of public water supplies had a detrimental

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effect upon brass pipe. It stands to reason that no one will go to the expense of applying corrective chemical treatment unless prior experience indicates that the pipe in the system is corroding; and if so, the question is, will corrective treatment improve conditions or will it make them worse. Apparently it will improve conditions in most cases. There are a great many ways in which corrective treatment may be obtained. Corrective chemicals can be added in the form of sodium carbonate, or in the form of lime; the two chemicals will act very differently. While both will raise the pH value of the water sodium carbonate will do so without forming deposit coatings, while lime may at the same time lead to the formation of deposit coatings. These deposit coatings in themselves may be the protective agencies rather than the higher pH values. Has anyone had experience with corrective treatment in which it is indicated that higher pH values increase the corrosion of brass rather than decreasing it.

MR. BEAN. I know of a certain water where that does happen; corrosion of zinc is increased by increasing the pH. In this case however, the increase of pH value produces a reduction in the corrosion of iron to such an extent as to make the increase in pH desirable, in spite of the fact that it results in increased corrosion of zinc.

E. SHERMAN CHASE.* The town of Falmouth for the last two or three years has been correcting the corrosive quality of its water supply, using lime. Prior to the full-scale application of the lime to the supply certain laboratory tests were run using pure iron wire as the control. These tests were made to bring about an accelerated effect of the corrosive water upon the iron. It was shown by the tests that if the pH was raised with soda ash above, I think, pH 9, the iron was corroded almost as fast as at the lower pH values. The laboratory tests indicated a distinct optimum pH at which the corrosion of the pure iron was at a minimum. In practice in Falmouth it has been customary, with the use of ordinary hydrated lime, to get a residual pH of 8.8 to 9. While this practice has not entirely overcome the corrosive effect of the Famouth water it has very materially reduced it.

HARRY W. CLARK.† I think that the paper is a mighty good paper on pure chemistry, but when you put pipes in solutions you sometimes get very different results in regard to corrosion than you do when working with pipes of copper or brass, with water on the inside of the pipe only.

The pH does have some effect, of course, on preventing or controlling corrosion—a considerable effect; but in a paper that I read before this Association in 1927 I stated, as some others have also stated, that pH was not the only thing; that certain constituents of the water also had a good deal of effect upon preventing or accelerating corrosion.

I have had personal experience with galvanic action or electrolysis in my own house within the past two years. I have had brass pipes in use for twenty-two years and never had any corrosion worth mentioning. Two

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years ago I put in an electric radio that was grounded on a pipe in the basement. Almost immediately I began to have blue-green deposits in the tubs and also in the set bowls in the bath-rooms. I was a little puzzled for a while in regard to this, not thinking of the electric radio, but on moving the grounding to a different place the blue coloration disappeared. It may have been something else you know that caused the corrosion but I laid it to the radio.

MR. HEFFERNAN. I received a letter last week from an official of a private water company located just outside of New York drawing my attention to the following facts. Blue water found in a bathtub was traced to a radio grounded twenty feet inside of the water meter. The blue, which apparently was caused by slightly disintegrated brass pipe, disappeared when the ground wire was moved to a different point. As Mr. Clark says, you can make all the studies you want in the laboratory but you will not get definite results with all the conditions that are causing the corrosion of the pipe.

Today the water-works superintendents are somewhat puzzled about the choice of material to use for service work that will be immune from all objectionable conditions. Galvanized wrought-iron and steel pipe lined and coated both inside and outside by dipping in a bath of molten zinc is the most commonly used form of metallic coating. The threads which are usually cut after galvanizing expose the bare iron to corrosive action of the water; galvanic currents passing from zinc to iron remove the zinc at the threads exposing large surfaces of iron to corrosive action. Copper and red brass pipes are used by many communities and from all reports are giving perfect satisfaction. I still doubt the permanent adoption of these materials for services underground. The objections being the discoloration of water caused probably by soil corrosion, galvanic action, electrolytic action, and grounding of wires.

MR. CLARK. I think you will get just as good results with copper pipe as with brass pipe.

MR. HEFFERNAN. I do not approve of either one underground. I am of the opinion that iron-sized brass pipe is the best to use inside the building. If you want to avoid some of these annoying elements in service pipes, especially the discoloration by blue or green water, use wrought-iron cement-lined pipe with special lined fittings. It should not be forgotten to line the curb cock at the sidewalk. This is the method used in Milton and it gives general satisfaction.

MR. CLARK. I think they both, brass or copper, and cement-lined pipes have their place. In the last few years all our experiments on corrosion have been done with pipes set up as service pipes,—copper, brass, lead, or whatnot—because in the old days when we used to immerse them, sometimes we would get very reasonable results and sometimes very unreasonable results.

I believe that Mr. Heffernan will, one of these days, perhaps, put in a brass pipe, or a copper pipe, and see that it works all right.

MR. SMITH. To answer Professor Fair's question, he knows that in Baltimore the water is filtered, treated with lime to reduce its corrosiveness. Very good results are obtained, the pH being adjusted at about eight, I believe. As far as I know, this treated water is not actually very corrosive. I do not know exactly the type of water which they treat but it has an important bearing on the results obtained. There is a small filter plant at Hampton, Va., where water is taken from a reservoir the basin of which, I understand, was originally a bog or marsh. The water was found to be acid and corrosive, and it was treated with lime until the pH was brought up to about nine, but the corrosion was not stopped. I believe that this is because this very soft water treated with lime is actually alkaline with normal carbonates; even though the pH value has been raised, the water is corrosive toward the cheaper grades of brass pipe. It appears from these two examples that different types of water cannot be treated in the same manner to raise the pH value to a high point with the result that corrosiveness is always reduced.

MR. FAIR. I have never believed that the solution of the corrosion problem lay alone in the increase of the pH to decrease the solution of the metal. It has always been my opinion that Nature's most important way to decrease corrosion is to form a protective coating on the metal. The purpose of artificial treatment may, therefore, well be such that a protective coating will be formed. If it is not formed there may be solution of the metal. In line with this reasoning I am more favorable to the use of lime rather than soda ash, because lime does, under proper circumstances, form protective coatings. A cement lining in a pipe is a form of protective coating. There may be situations where no reasonable kind of chemical treatment will form a protective coating. There we have to resort to protective coatings in the form of cement linings.

As to the difficulty of interpreting laboratory studies, such as Mr. Moore's, in the light of experience, it is my belief that theory must go hand in hand with practice if we are to arrive at a broad solution of this rising problem of corrosion. Pure chemistry has its place in this scheme as well as pure empiricisms. In the long run theory will explain what practice finds to be true.

MR. MOORE. The discussion of the author's paper has clearly shown the difficulties of applying information on corrosion obtained in the laboratory to the phenomena of corrosion under service conditions. In the latter case, the extent of corrosion is determined by a great many factors, among which are the mineral content and dissolved oxygen content of the water, the presence or absence of protective coatings, and the influence of stray electric currents. These factors may also vary from time to time within a given system. Therefore the laboratory worker's possible service is limited to the determination of the quantitative effect of one or the other of these factors when all the rest are held constant. He cannot decide with any certainty whether a water will or will not be seriously corrosive. He can only

state under what circumstances its corrosive power will be reduced or augmented.

The purpose of the paper was an attempt to show what happens to brass when the pH alone varies and all other factors are held constant, or eliminated. The deductions drawn are therefore strictly relative to the known behavior of any water to which they are applied. For example, if a water is highly corrosive to begin with, the reduction in corrosion obtained by merely raising the pH may be entirely negligible from a service standpoint. Another water may be so kind to the piping which carries it, that there is no point in raising the pH.

The case cited by Mr. Bean makes it evident that the minimum corrosion point for zinc is not always at pH 10.0. In that case it was evidently at or near eight. If the author's tests had continued for a long time, it is possible that a pH of 8.0 or 9.0 might show as good or better results than pH 10.0 by virtue of the greater stability of the protective coatings of zinc compounds at the lower pH values. This fact was stated at the time the paper was presented.

A HISTORY OF WATER SERVICE INSTALLATION IN NEWTON.

BY MICHAEL J. HERLIHY.*

[*Read December 18, 1933.*]

The beginning of the Newton Water Department dates back to the early part of 1876, at the end of which year, there had been installed 496 services. At the present time, there have been installed some 15 000 services. During the fifty-six year interval in which the Department has been engaged in supplying water to its consumers, it has had some varied experiences with different types of service pipes.

In the early years the practice of using black iron pipe for services was followed. The sizes used were ¾, 1½ and 2 in. and a tap one size smaller than the pipe was employed on all except the 2 in. services on which it was the practice to utilize a ⅝-in and 1-in. tap, yoked. All taps were connected to the service pipe by a lead gooseneck of the same size as the tap.

Later the use of black iron pipe was abandoned and lead pipe was substituted, the sizes selected being, ⅝, ¾ and 1 in. The Department reserved the right to specify the kind and size of pipe to be used although sometimes the Department gave way to the owner and allowed him to buy and lay his own pipe from the street line to the cellar. In later years, if the owner insisted on laying his own pipe, a meter was installed in a box at the street line, on end of the pipe laid by the Department. Today the Department furnishes and lays the pipe and sets a meter in the cellar in all cases except where there is no cellar under the building. Then the meter is set in a meter box at the street line.

A large percentage of the houses during the early history of the Department did not have any heat in the basement, and a great many services were laid through the wall and under the cellar floor which frequently had only a dirt surface. Hence when the meters were set, it was customary to bury them under the floor in a small box. This resulted in a poor setting from two standpoints: (1) it was not simple to replace the meter for any reason; and (2) considerable difficulty was experienced in reading meters so placed, as wood and other materials generally found their way to that point.

The use of lead pipe was continued until about 1900, when presumably due to the number of leaks occurring in lead services, this practice was dropped and the Department then changed to 1-, 1½- and 2-in. galvanized wrought-iron pipe as a standard. One-inch taps and goosenecks were used to connect to the 1-in. services. One and one-half-inch taps and rigid connections were used on 1½-in. services. The 2-in. services were fed by two

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taps, a 1½-in. and a 1-in. tap. The larger of these taps was connected by a rigid connection to the service pipe while the smaller was connected by a 1-in. gooseneck to a 2-in. by 1-in. tee.

During the period between 1910 and 1919 the Department experienced a great deal of trouble with the splitting of lead goosenecks. In 1919 the Department was using a 1-in. Corporation cock with a female outlet. It was decided then to make rigid connections to the taps, replacing the gooseneck by a 1-in. galvanized nipple and a 1-in. union and to lay the pipe on solid ground to the cellar. The Department was very careful in the preparation of the foundation for the pipes and insisted on building up under all couplings and fittings and using great care in back-filling and packing under the pipes so as to have no strain on them. This practice was felt necessary to insure the success of a rigid connection, as had been learned by experience since 1900 on 1½-in. service installations.

From this time on the Department steadfastly refused to permit any other utility to lay pipes within four feet of the water-service pipe. Where a sewer connection must be laid in close proximity to the water pipe the sewer must be laid first and the sewer trench widened from a point not less than 4½ ft. below the surface to form a shelf of solid material. On this shelf the Water Department will lay its pipe. Lead goose-neck connections were no longer used except in swampy or wet ground and wherever required in the repair of old installations.

The Department wherever possible required that the service pipe be entered through the foundation above the cellar floor, so as to be able to make a proper meter setting. Also in case of corrosion at the tap or the curb cock, this practice permits of the clearing of the service and eliminates the cost of digging up either the basement floor or the street. In cases where insufficient cover existed over pipe at the grade outside the cellar wall the pipe was brought in under the cellar floor, then up a distance of six inches above the floor.

Although the Department felt that it had a fairly good service installation at this time, a weakness developed, in that the 1-in. galvanized nipple corroded at the threads where it was screwed into the tap thus causing leakage. The same thing happened at the curb cock which at that time had female outlets. Therefore a heavy copper nipple was substituted at the tap and one was used on each side of the curb cock. This practice worked out very well but the objection was offered that the use of three different metals in close proximity one to each other might induce galvanic action. While our experience has since lead us to question the seriousness of any damage which might result from such action between dissimilar metals, the Department about 1925 adopted a corporation cock using a union tail piece of the same metal as the cock body and also curb cocks with male outlets.

In 1929 the Department made its next change in service pipe practice, changing from galvanized to cement-lined wrought-iron pipe with lead-

lined fittings. It was decided to use only two sizes: 1¼ and 2 in. This decision the Department arrived at after taking into consideration the type of residences being built in Newton, the set backs ordinarily established and types of plumbing fixtures being installed. Newton is zoned in such a manner that 92 per cent. of the city is reserved for residential property, and the usual set backs are not less than 30 ft. Within the last few years most of the properties constructed have had two or more full baths, and in a great many cases flushometers have been installed. The demand resultant from the construction of properties of this kind is for a large volume of water in a limited period of time and it was deemed wise to reduce as much as possible the friction losses in the service pipes.

The method of installing services up until 1931 was to take sufficient pipe to the job and there to cut, thread and fabricate the line to meet the requirements. This practice proved more or less unsatisfactory inasmuch as considerable difficulty was experienced in keeping the tools and dies sharp. Too frequent recourse was taken to pipe cutters that caused the cement lining to shale off exposing the unlined metal. Each service truck became more or less cluttered with small ends of pipe,—from a few inches to three or four feet in length,—most of which became badly battered and a large proportion of which found its way into the junk pile.

In 1931 the Department's supply yard was rebuilt and there was provided in our remodeled structure an up-to-date pipe shop. Modern pipe-cutting and threading machinery was installed and the service pipe construction methods were again modified.

The present procedure is to attempt to so lay out our service work, that the main to be tapped will be exposed before noon. The chauffeur assigned to cover any particular service installation or installations is required to visit his jobs between 11 A.M. and 12 NOON and to receive from the piper the measurements of the pipe required. These measurements are turned in at the supply building at noon and during the lunch hour the man on duty cuts, threads and fabricates complete the pipe as requisitioned. All couplings and fittings are run up on the threads but are not "put home" until they are in the trench. All pipe is made ready by 1 P.M. for delivery to the different jobs by the chauffeurs. The field piper then has only to put together any long lengths and to fit the pipe into the trench.

This practice assures the use of smooth and evenly cut threads, properly fitted couplings and fittings, takes less time and produces a thoroughly workman-like job. Furthermore the shop man has an opportunity to utilize completely all small pieces of pipe so that all waste is eliminated.

In order to eliminate some breakage of curb cocks which had been experienced where heavy trucks pushed our service boxes down on the fitting, the Department in 1932 adopted a 1-in. curb with heavy 1¼-in. male outlets. This provides a minimum metal thickness of ¼-in.

The Department's records of the last two years show a total of 384 service leaks, or an average of 192 per year. Fifty-eight and one-half

per cent. of these were caused by split lead goosenecks, 11.2 per cent. were caused by corroded 1-in. galvanized nipples at the taps or curb stops, the remaining 30 per cent. of the leaks were due to various other causes.

With approximately 15 000 services in use today the total number of leaks occurring in an average year amounts to 0.64 per cent. This figure we believe will be materially reduced as we gradually replace the gooseneck connections and the 1-in. galvanized nipples at the corporation and curb cocks (which as before stated are responsible for the bulk of our service leakage) and as the proportion of services using rigid connections, now 40 per cent. of the total, increases with our growth.

Discussion.

Roger W. Esty.* We have been using goosenecks in our service connections for a good many years but have not run into any trouble.

What is the pressure at Newton?

Mr. Herlihy. Our pressure varies. As an average over the whole system it is between 60 and 70 lb. per sq. in. Our greatest pressure is around 135 lb., and it goes down to 35 or 40 lb.

Henry T. Gidley.† I would like to ask Mr. Herlihy what is the weight of the lead used for goosenecks?

Mr. Herlihy. It is 5½ lb. per ft. for 1-in. pipe, and 4½ lb. for ⅝-in.

David A. Heffernan.‡ I think that on a ⅝-in. pipe the weight of lead pipe should be 3 lb. per ft. for good water-works practice. On ¾-in. pipe, to take in all pressures from 40 to 130 lb. per sq. in., it should be 5 lb. On 1-in. pipe I use 6 lb. per ft. and on the 1¼-in pipe 9 lb. per ft.

Mr. Herlihy spoke about having a lot of trouble from split lead pipe. You never can get split pipe with such weights.

Nelson Boardman.§ I would like to ask Mr. Herlihy whether he uses flanged joints on the gooseneck?

Mr. Herlihy. No. It is a special fitting which was devised years ago by one of the men connected with the Newton Water Department.

Mr. Boardman. Is it soldered in?

Mr. Herlihy. It is fitted with a thimble, into the lead.

Mr. Esty. What kind of a connection do you have with the outside meter?

Mr. Herlihy. We have a cement meter box which comes in four sections. When we set the meter we carry it to within a foot of the top of the box, so it is easily set and easily read.

Mr. Heffernan. I do not think that the water-works officials take things seriously enough relative to just what is causing the trouble in their service pipes, or main pipes. There are three elements, in my opinion,

*Superintendent, Danvers, Mass. †FairhavenWater Co., Fairhaven, Mass.
‡Superintendent, Milton, Mass. §Assistant Superintendent, Danvers, Mass.

which I consider important in deciding how we shall connect up our service pipe to the main. These are soil corrosion, galvanic action, and electrolysis.

Soil corrosion: Soil corrosion of underground pipe is caused by action of chemicals in the soil and ground water and cannot take place in the absence of moisture. Temperature, soil characteristics, conductivity, and stray electric currents all affect the rate of corrosion.

Galvanic action: Galvanic corrosion of water pipe is caused by contact between pipes and fittings of different materials, such as clamps, U-bolts, gaskets, corporation cocks, wrought-iron risers on cast-iron services, lead goosenecks, impurities in the metal itself, poor application of metallic coatings, close proximity of pipe lines underground, and use of unsatisfactory materials for back-filling trenches.

Electrolysis: Electrolysis, or electrolytic corrosion, occurs in water pipes when an electric current of external origin—usually stray direct current from the return circuit of a street railway system, or electric power circuit whose rails are not properly bonded—gets into the water pipe. There is also the grounding of wires on house service pipes with alternating current to provide a metallic return circuit for the current, and thus improve the quality of the service without the expense of return wires. These disturbing conditions I honestly believe are not taken seriously enough by water-works men to obtain the best standards in service installation.

Many water departments have abandoned the lead gooseneck in connecting up their services, one of the reasons for doing this, in my opinion, being to eliminate the possibility of leakage at the joints, cup or wiped. From my own experience I believe if the joints are made properly and heavy lead pipe is used, there will be no trouble from this source. Another reason for not using the lead gooseneck is that it does away with the services of the skilled mechanic and permits the substitution of the handy man in his place. The advantage of the lead gooseneck over the rigid joint connection allows a margin of safety for expansion and contraction, settlement of materials, etc.

Conditions in trench excavating are different today than they were twenty years ago. Permanent road construction requires other utilities to locate their pipes in one street opening. I do not approve of this method. But nevertheless it is being done in many places. This means the supporting material under the pipes will be disturbed. With this condition the lead gooseneck with its margin of safety will stand up better than a rigid joint.

That to my mind is a very serious condition. Mr. Herlihy spoke of different services being 4 ft. apart. That requires a pretty wide opening. He also mentioned putting waterpipe on a shelf. That shelf, I suppose, will be about 1 ft. The gas line, I suppose, would be placed on one side on a shelf and the water on the other side. Is that the procedure?

MR. HERLIHY. We generally try to keep as far away from the sewer trench as possible. We insist that we lay our pipe on solid ground right from the main to the cellar.

MR. HEFFERNAN. If your practice could be carried out it would be a very good practice, but we can't do it in Milton. With your good paving the street department will only permit us to make one opening, and the pipe must be placed in that trench on a shelf. If you have a rigid joint and if there is any settlement of the material, there is going to be a strain and something has to go.

There is one thing more I would like to say. Substantiating my remarks relative to electrolysis, it might be of interest to the members to report from the Plumbing and Heating Trade Journal for the month of November, 1933, an interesting article by A. R. McGonegal, Chief Plumbing Inspector, District of Columbia:

> The American Water Works Association, as the only organized representative of the hundreds of municipal and privately owned public water supply systems, gave away something many years ago to which it had no title and in which it had no direct financial interest.
>
> The compilers of the National Electrical Code demonstrated a supposed need for grounding a neutral wire of an electric circuit between the main circuit on the poles and the terminals at the main switch on the customer's premises. It was speciously pointed out that a neutral normally carried no current, and in emergency only a very small potential. Under such circumstances, the grid formed by the public water mains under the streets became an ideal ground for occasional stray currents and, by reason of its grid form and its house-to-house connections, would act not only as a ground, but also as a balance.
>
> Thus, the compilers of the National Electrical Code calmly accepted something that the American Water Works Association had no right to give, and forthwith put it in the draft of the Electrical Code, where it has been ever since, frequently to the detriment of the public water supply distributing systems. Some electric service utilities even went so far as to run bare neutral wires, thus saving cost of insulation, though this practice is forbidden by local rules in a majority of cities.
>
> Now there is a concerted and well-planned movement on foot to extend this questionable practice to electric light and power wiring on the interior of the consumer's premises, placing the burden of carrying any stray or accidental return electric currents on the pipes of the plumbing system, and the master plumber, who is responsible for the integrity of the plumbing system, has been given no voice in the matter.

FRANK S. BRAINARD.* I would like to ask how Mr. Herlihy makes the rigid connection; what fittings are used in the connection.

MR. HERLIHY. Insert a tap in the side of the main with the union tailpiece, which is secured into a fitting. Where we use a 1¼-in. pipe we have a 1¼ by 1-in. fitting, then the pipe from that point is straight into the cellar. There are no other fittings. All couplings are built up, and all fittings are packed very carefully underneath.

MR. GIDLEY. I think Mr. Heffernan brought out some very good points. I think the great trouble in lead pipe and lead goosenecks lies, as he said, in the weight of the pipe. We use for ¾-in. pipe, 4-lb. lead, and about 6 lb. for 1-in. pipe.

He also spoke about the grounding of wires to the waterpipe. That is a thing I think people have not looked into enough but are looking into

*Hartford, Conn.

it now. We ground the telephone, the electric light and the radio on the waterpipe. In certain experiments it is reported that trouble was experienced with colored water in certain sections, and they also had trouble with the taste of the water. They tried a number of experiments, and when they disconnected the radio from the waterpipe the water cleared up, had a good taste and was not colorecd.

A MEMBER. In my town we have been conducting a series of experiments for grounds. We have been using Leadite or Lead-Hydro-Tite for the last fourteen or fifteen years, and the electric services have practically come to the decision to do away with pipe connections in our town.

EDMUND C. SANDERSON.* I have been in the electrical business for thirty years, and I have followed the grounding pretty carefully. We have never had a case in Winchester where our pipes have been damaged from the alternating current escaping to our system. Alternating current does not cause electrolysis; direct current does. Since we have changed all our street railways we have not had any more trouble from electrolysis.

I think this subject is misunderstood. The grounding of house currents is a safety device and I think it should be practiced. We go so far that when we put in a new service we make a charge of $1.50 to see that the ground cap is properly put back. From our experience we have not had any trouble whatever with it. We use brass pipe and lead pipe, and our old pipe is galvanized iron. We are fortunate enough to have the water and the sewer services in one department. We find it works very well. We always lay the waterpipe with the sewer, in the same trench. Our men are careful in seeing that the waterpipe has a proper footing, and we have never had any trouble with it. We use recess couplings and have practically no service trouble whatever.

MR. BRAINARD. We use a swing or semirigid connection. We have had practically no trouble since we switched to that, and in our opinion we much prefer the semirigid connection which we use to the lead connections which we formerly employed.

MR. HEFFERNAN. Is that on a street L?

MR. BRAINARD. No. On the ordinary corporation cock, screwed into the main, and then the street L. There are two street Ls and the ordinary street L used at the main, making a swing connection there which is not rigid, which would allow for some settlement.

MR. HEFFERNAN. I would like to ask Mr. Herlihy a question: Where do you tap a pipe; at an angle of 45 degrees; on what part of the main?

MR. HERLIHY. Practically the center.

*Chairman, Water Board, Winchester, Mass.

WATER WORKS PRACTICE AT TAUNTON, MASSACHUSETTS.

BY ARTHUR C. KING.*

[*Read December 18, 1933.*]

It is not the intention of this short paper to hold up the practice of the Taunton Water Works as an ideal or model, but rather to describe what is being done, in the hope that it may be helpful to some member, either directly as an idea to be improved upon, or only by elimination, suggesting something that might better be avoided.

Taunton is a city of only about 37 000 people, but covers an area of 50 sq. miles, being one of the largest cities in the country in order of area. This is of course a disadvantage rather than otherwise to all utilities in the added expense to reach and serve a scattered population. The water system has always been municipally owned since its establishment in 1876. Its management is in the hands of a Board of three Commissioners, serving three-year terms, one member being elected each year by the City Council. The Board organizes by choosing one member as President and a Clerk who is not a member. The Clerk's salary, as well as that of the members of the Board, is fixed by ordinance, but the Board fixes the compensation of all other employees. The Clerk's duties include those usual to the office of Registrar, but not of Superintendent, although it has been customary since early in the history of the works to employ the Clerk also as Superintendent.

The source of water supply is a chain of five connecting natural ponds with a watershed of about 50 sq. miles. These ponds also supply the city of New Bedford. Under Chapter 400 of the Acts of 1924, division of expenses and allotment of water were provided for between New Bedford, Taunton and Fall River, but since that year Fall River's need for additional supply has disappeared. Taunton's allotment is 8 m.g.d., but the average daily consumption has not yet been more than 3¼ m.g.d. and for the last few years has been below 3 m.g.d.

Swimming is forbidden in the ponds from which Taunton and New Bedford draw directly, while boating and fishing are allowed by special permits issued by the two Water Departments. The water is used untreated, and Taunton does not regularly patrol the shores or watershed although in the writer's opinion it would be advisable, in the summer at least. Dependence for purity is placed upon long storage and occasional inspection of conditions near the shores. Water samples are examined by the State Department of Public Health each month. From the largest of the ponds, Lake Assawompset, which has an area of 2 178 acres and an estimated capacity of 2½ billion gallons, water is pumped through a 24-in. cast-iron main about 1½ miles long into Elders Pond, a spring-fed pond

*Superintendent and Clerk, Taunton Water Works, Taunton, Mass.

about 145 acres in area with a capacity of about 692 m.g., from which the water flows by gravity through a 30-in. cast-iron main 7½ miles long to the city.

The pumping is done by a 5-m.g.d. vertical, triple expansion steam engine installed in 1894. Through the summer this pump is normally operated 24 hours per day, six days a week, but in winter only as necessary to keep up the water level in Elders Pond. When the intake water at Assawompset is objectionable due to vegetation or because it is stirred up by a strong east wind across the pond, and also when young herring are seeking the outlet, pumping is discontinued until conditions are favorable. Such periods are used for repairs and overhauling of the pump. Two or three men are constantly employed around the shores and on the watershed mowing, ditching, cleaning intake screens, trimming trees, etc. When regular employees in the city are available they are sent over for a day or more at a time for this work. In winter, if the ice becomes thick, a lane in the ice is cut as often as necessary to keep the ice pressure from damaging the gate houses or the riprap on the intake dikes. Once or twice a year the grass and water plants growing within a quarter mile from the intake are mowed with a submarine saw 50 ft. long and the plants raked from the water. Considerable portions of the land owned on the watershed have been set out with trees, mostly white pines. There is a considerable number of summer cottages around the shores, but most of the owners are ready to coöperate in keeping sanitary conditions satisfactory. If property on the shore comes on the market at a price which can legally be paid it is the practice of the Department to purchase it, and if any owner neglects or refuses to comply with sanitary regulations he is liable to wake up some morning and find that his property has been taken from him.

Entirely around Elders Pond a strip of land 5 rods wide is owned, as well as some much larger tracts on the watershed adjoining. The whole shore, which is sandy, is harrowed and raked to keep down all vegetation.

The 30-in. main from Elders Pond delivers the water at a pressure of about 25 lb. per sq. in. to the distribution pumping station, which is about 1½ miles east of the center of the city. There is no elevated storage, so this station has operated without interruption for 57 years. This of course necessitates duplication of all essential parts. There are three pumps, each of which can take steam from either of two steam lines from the boilers. Two boilers, half banked, are on the line at all times, with a third in reserve, but cold, although one could carry the normal load. While this is not economical it is necessitated by the possibility of large demand for fire fighting at any moment. At every alarm of fire the pump is speeded up a little, to raise the water pressure from 70 lb. per sq. in., the domestic pressure, to 100 lb. per sq. in. fire pressure, or to 105 or 110 lb. for second alarms from certain boxes in higher sections of the city. This changing of pressure is a source of annoyance, or worse, by developing leaks just when

they are least desired, that is, when both volume and pressure are wanted by the firemen.

As an illustration I shall mention a break that occurred in 1932 at 2 o'clock in the morning of St. Patrick's Day. The pressure had been raised for a small fire, a hen coop I believe, on which one stream was playing when suddenly the pressure dropped, and the engineer at the pumping station thought for a moment that the supply line had burst and left him without water. Shortly after, police reported water coming up through the concrete paving on Main St., and after shutting down the 16-in main and opening the pavement, it was found that one length of cast-iron pipe, in use about 56 years, had suddenly cracked nearly its whole length. Pressure could not be maintained in the rest of the system until this section was cut out. Continuous work for 24 hours was necessary to replace this length of pipe and restore the service on one side of Main St.

The main pumping unit is a horizontal cross-compound engine of 10-m.g.d. capacity, installed in 1914. Since that time this pump has done the bulk of the work, as it is economical and very flexible. Its speed can be varied from 46 r.p.m. for fire draft down to 8 or 9 as the consumption falls off in the early morning hours. The other pumps are both horizontal compound engines, one with a capacity of 8 m.g.d. installed in 1901, the other rating 4 m.g.d. installed in 1891. Steam is always in their cylinder jackets keeping them warm ready for use, and each of them is operated for several hours once a week. In case of failure of the supply from Elders Pond, either the 10-m.g.d. or the 4 m.g.d. pump can draw for a few hours from an infiltration gallery at the rear of the station, formerly a source of supply.

From this pumping station two 20-in. mains follow different routes toward the center of the city and the gridiron system. From one of the 20-in. pipes a 16-in. main leads to the north end and a similar 16-in. main leads from the other 20-in. pipe to the south end. Interlocking loops of 12-in. pipe completely surround all built-up sections of the city, with 8-in. secondary feeders within the loops to 6-in. lines. Little 4-in. main is still in use since 1930 when a five-year program of replacement was completed. Twenty-five miles of 8-in. and 12-in. mains were laid in that program, replacing 4-in. and 6-in. mains, many of which had been in use 45 to 55 years. This program cost $435 000 which might have been spread over 25 years instead of 5 if the Water Board had had control of its receipts. Some of the mains taken out had been recommended for replacement at least 20 years before the inauguration of this program. All of the pipe used in the five years was centrifugally cast and cement lined. Fittings were also cement lined.

One man is held responsible for the condition of hydrants. He uses a light truck and upon occasion is given help temporarily, as in very cold weather, for heavy repairs or to speed up the fall preparations for cold weather if it becomes necessary. Every hydrant is operated and tested at least twice a year, in the spring and again in the fall, and during cold

weather it is tested at intervals for dryness to prevent freezing. Denatured alcohol is placed in those hydrants which cannot be kept dry, but if one does freeze it is thawed with lump carbide.

A foreman answers every fire alarm and notes what hydrants are in use so that the hydrant man may check them the following day, and the Fire Department reports any hydrants used when no alarm was sounded.

In summer, hydrants are opened each morning and closed at night for construction work by other departments or by contractors, a valve being placed for their manipulation through the day. Meters are also set with these valves where there is to be any considerable use of water.

Dead ends are flushed regularly in summer, and upon complaint in the winter. It is planned to inaugurate a system of gate inspection next spring.

From the beginning of the works in 1876 until 1927 wrought-iron pipe lined with Rosendale cement by Department employees was used for service pipe, with lead goosenecks. Since then, red brass pipe, iron-pipe size, has been used, connected rigidly to the main with taps made horizontally. The standard is 1-in. throughout, although 1½-in. and 2-in. pipes are also laid of red brass pipe. If a pipe larger than 2 in. is required a 4-in. or 6-in. cast-iron pipe is laid. The Department installs the service from the main into the cellar, charging the owner for the part on his property only, at a standard rate per foot, 50 cents for 1-in, 80 cents for 1½-in, and $1.00 for 2-in. pipe. The Department maintains the service without charge but does not maintain or install the shut-off in the cellar. Meters are paid for by the owner but are maintained free of charge.

The maintenance of the 6 600 services requires the employment of three men with a light truck throughout the year, with extra help for digging. Poor pressure complaints to the number of 500 or 600 per year are largely remedied by wiring; that is, pushing and twisting a heavy wire through the pipe from the cellar, with the water turned on to wash out what is loosened by the wire.

Meter work keeps six men busy, two men with a truck taking out and resetting meters, two repairing meters and two reading them. Every meter is cleaned and tested at least once in four years. Even though a meter shows no indication of anything wrong, if it has been in service without a test for four years it is brought in, tested, cleaned, repaired if necessary, and tested again, and made to register between 98 and 100 per cent. on three or more different sized streams, according to the size of the meter. The smallest stream regularly used is ⅛ in. Meters are read once in about five weeks, although three in heavy use are read weekly. When the domestic bills are being made, in June and December, all of the meter department except the man in charge, five in number, read for two weeks or more to cover the whole system as quickly as possible. The regular meter readers, when not doubling back to cover "outs," average about 120 readings per

day. Upon complaint of large bills, fixtures are examined for leakage without charge.

As was mentioned earlier in this paper, the duties of the Clerk of the Board include those commonly taken care of by the Registrar; that is, the accounting of the Department, also billing and collecting. Receipts are turned over to the City Treasurer, and all bills and payrolls of the Department are paid by him after approval by the Board.

Operating funds are appropriated by the City Council, as they are for other departments. There are no complete bookkeeping or cost account systems as the office force has been kept at a minimum. In addition to the Clerk of the Board, who is also Superintendent, there are three office clerks who subtract the meter readings, make out the bills, post the receipts, and keep up the usual office work. Domestic consumption is billed semi-annually, business quarterly. An Addressograph is used, but aside from that the bills are made out by hand, as well as the postal card notices which are mailed, the bill and cashier's stub remaining in the office until paid.

While the meter readings are being subtracted in the office after each reading, whenever an unusually large use is noticed, unless a reason is indicated, a postal card notice of increased consumption is sent to the owner as a warning of possible leaks unknown to him. Also, if the reader has been unable to get in for three consecutive months to read the meter, a card is sent to the owner asking that arrangements be made for the reading to be secured. A ledger card is used for each service pipe. This is a change made recently from the use of a book in the hope of purchasing a billing machine before long, with which the cards would be needed. The card system entails a little more work now by hand, but will save a great deal when employed in connection with the machine.

When a water bill remains unpaid after twenty days, a second notice is sent which brings in quite a number of people. Although the lien law has been accepted by the city, it has not yet been used. Occasionally water is shut off on account of non-payment of bills, but chief dependence is placed upon personal insistence. This is one feature of the customary procedure in Taunton which I wish to emphasize as one to be avoided. I believe that nearly all can pay their water bill as they should with reasonable promptness, and they will do so if an inflexible system is used, always shutting off the water after a definite time for payment has passed. If people realize that service actually will be discontinued the money will be forthcoming. Without strict adherence to such practice, the same people will again and again neglect or put off payment of every bill as it comes along. Of course this does not apply completely under the conditions of the past two years. Anyone who has come in and said he was unable to pay from lack of work has been urged to bring in any amount, even 50 cents or $1 when he could, and it is credited to him. A great many have taken advantage of the offer and appreciated the accommodation.

For unemployment relief the City Council has made three special appropriations by bond issues, amounting to $30 000, with which the Water Department has laid about 4½ miles of 6-in. main extending into outlying districts where the volume of business is too small for extensions to be made on the regular guarantee basis of $20 a year income from each 100 ft. of extension. In this work, pipe was laid that had been taken out in the five-year program previously mentioned, so that most of the expenditure was for labor. During spare time, regular employees have cleaned and sand-blasted the pipe and spray painted it with bitumastic paint, ready for use. The $30 000 has not been entirely spent, but this week two projects were started with C.W.A. funds, laying nearly a mile more of the 6-in. pipe. The pipe is to be laid by regular employees on the regular payroll, but the digging is being done by laborers on the C.W.A. payroll.

Discussion.

Roger W. Esty.* I would like to ask Mr. King to explain in more detail how he thaws out hydrants.

Mr. King. In cold weather the hydrant man carries a little lump of carbide, the same as we use in a carbide lamp. He breaks off a lump that will pass the nozzle of the hydrant, and the carbide will bore a hole through the ice in a few minutes. Then he can turn the water on in the hydrant and wash out the rest of the ice. It is much handier than trying to get hot water or steam into the hydrant.

Mr. Esty. Suppose you had a hydrant frozen from the top right down to the base. How long would it take to thaw it out and would it take more than one lump of carbide?

Mr. King. It probably would take several lumps. We do not often have them freeze as much as that.

*Superintendent, Danvers, Mass.

RESULTS OF WATER WASTE SURVEY IN LEXINGTON, MASSACHUSETTS.

BY A. A. ROSS.*

[*Read December 15, 1933.*]

The Town of Lexington receives its water supply from the Metropolitan District Commission. The water is supplied by gravity from a standpipe located on Arlington Heights and flows through a 16-in. cast-iron main to the Metropolitan Venturi meter in Massachusetts Avenue at the Lexington-Arlington line. At this point the water enters a 16-in. Lexington trunk main, which extends for a distance of approximately 3 miles to the standpipe.

There are approximately 69 miles of mains in the distribution system. All the mains are cast iron, varying in size from 4 to 16 in., with a very small amount of 4-in. pipe. A concrete standpipe 105 ft. high and 30 ft. in diameter with a capacity of 550 000 gal. is connected with the system. The population supplied is approximately 11 000.

In the fall of 1931 an investigation was made of the consumption of water as recorded by domestic and industrial meters. This investigation disclosed that only 53½ per cent. of the total consumption as recorded by the Metropolitan Venturi meter was accounted for. An attempt was made by the Department to locate possible leaks, but we were unable to find any of a serious nature. It was then decided to have a survey made, and a contract was entered into with the Pitometer Company of New York in the late fall of 1931. Work was started immediately and two buildings 6 ft. square were constructed to house the instruments, and a crew of four men and a small truck were supplied to work with the Pitometer engineer in charge.

To measure the flow of water in mains buried in the streets, an instrument known as the Pitometer is used. It includes two slender, metal tubes, so fashioned that they can be introduced into a main through a "tap" similar to that made for a house connection. The end of one tube points upstream; that of the other downstream. Greater pressure is thus created in one tube than in the other. By connecting the tubes with the ends of a U-tube of glass containing a colored measuring liquid, the pressures in the tubes are transmitted to the colored liquid, and the greater pressure depresses the liquid in one column of the U-tube. This difference in the columns is a measure of the velocity of the water, and the variations are recorded photographically on a chart. The velocity being known, the volume of flow can be computed.

*Superintendent of Water and Sewer Department, Lexington, Mass.

The scope of the survey made by the Pitometer Company consisted of dividing the town into five districts which were supplied with water through a single main and the flow into each area was measured with a Pitometer for 24 hours. To account for the minimum flow at night, when most water-using activities cease, is the problem. Therefore the survey proceeds to segregate first one block in an area and then to expand block by block, studying the increase in flow as each block is added. When a block shows an excessive rate of flow, an investigation is made with instruments which enable the investigator to hear the noise of water escaping from the leak.

Total Consumption.

The total consumption was measured by a Pitometer on the 16-in. main over a 24-hour period. In order to have all the water supplied to the town pass through this main, it was by-passed through a 10-in. main. The results of the tests were as follows:

Total Consumption	695 000 g.p.d
Maximum Rate for One Hour	1 000 000 g.p.d
Minimum Night Rate	352 000 g.p.d.
Ratio-Minimum Night Rate to Total	51 per cent.

District Measurements.

District No. 1. This district contains approximately 16 miles of main, and its character is middle and high class residential. It was isolated by closing several valves and the Pitometer measurement of the flow of water into this section was supplied through a 6-in. main. No leaks were found in this district.

District No. 2. This district is chiefly middle class residential and also includes part of the business section in the center of the town, consisting of approximately 20½ miles of water main. Two leaks were located in this district. One was a joint leak on an 8-in. main which was discharging 20 000 g.p.d. into an old drain; the other was a leak in a service that had been discontinued some 20 years ago. This service had been cut off on the house side of the curb, and it was found that the ¾-in. lead pipe leading from the main to the curb was split. The water from this leak was discharging into a catch basin nearby. The discharge from this leak amounted to 15 000 g.p.d.

District No. 3. This district is middle and high class residential consisting of approximately 10 miles of main. No leaks were found in this district.

District No. 4. This district is composed of middle class residences and the Middlesex County Hospital and includes approximately 8½ miles of water main. One leak was located in this district, a joint leak in a 4-in. main discharging 30 000 g.p.d. This leak was under a telephone conduit and was discharging into an underdrain which was used during the con-

struction of a trunk-line sewer and which was approximately 4 ft. deeper than the water main.

District No. 5. This district is middle and high class residential consisting of approximately 14 miles of main. Two leaks were located. One was a joint leak on a 4-in. main discharging 30 000 g.p.d. The water from this leak seeped through the ground a distance of approximately 1 000 ft. into a brook. The second leak was rather a difficult one to locate. It was found to be in a joint on a 10-in. main which had been cut off and capped. Two iron bars had been driven into the ground with several boulders of half-yard size to hold the cap in place. The cap had pulled away from the pipe about 2 in. causing a leakage of 50 000 g.p.d. The water in this case found its way through the ground for a distance of 10 or 20 ft. and was discharging into a surface-water drain.

Test of Standpipe, Meters and Valves.

It was not possible to obtain a direct measurement of the leakage through the standpipe by means of the Pitometer. A measurement was obtained by shutting the standpipe off and noting the drop of water level by means of a pressure gauge. Several tests were made in this way over short periods of time but the drop in the water level was not sufficient to record accurately on the gauge. However, from these several tests the leakage appeared to vary from 3 000 to 5 000 g.p.d.

All meters 3 in. in diameter or larger were tested for accuracy in place under normal conditions. All were found to be registering correctly with the exception of one 4-in. meter supplying water to a large estate, which was found to be under-registering 20 per cent.

The valves of the distribution system were found to be in good condition. During the course of the survey two valves were found to be closed, but they placed no serious handicap on the system except by creating undesired dead ends.

Summary.

During the course of the survey 5 underground leaks were located totaling 145 000 g.p.d. The industrial consumption amounted to 50 000 g.p.d. and this with the standpipe leakage made a total of 198 000 g.p.d. definitely accounted for by the survey. The balance represented the actual use by the people and the unavoidable leakage which exists in every water system. These were too small to justify the expense of locating and stopping them.

The Metropolitan Commission for the year in which this survey was made reported a total meter registration of 236 427 000 gallons, for which the Town was charged $22 688. Two-thirds of this charge, or $15 124 is for the actual quantity of water used, and is at the rate of $0.064 per thousand gallons. The leaks located which were wasting 54 000 000 gallons a year, at the above rate, cost the Town of Lexington $3 456 for the year.

	Gallons	Per cent
Purchased from Metropolitan	236 427 000	100
Sold through meters	126 760 000	53.5
Underground leakage (145 000 g.p.d.)	52 925 000	22.3
Leakage in standpipe (3 000 g.p.d.)	1 095 000	0.5
Unaccounted for (slippage and stoppage in small meters, public uses, unavoidable small leaks in 69 miles of mains)	55 925 000	23.7
Total Use	236 427 000	100

The total cost of this survey including the wages of men supplied by the town amounted to $2 831.55. The time required for the survey was 44 days.

Discussion.

Patrick Gear.* Of what material were the joints that blew out?

Mr. Ross. They were all lead joints. This is my eleventh year in Lexington, and all extensions during my time have been made with compound joints. No leaks were found in compound joints.

David A. Heffernan.† At Milton, with 81 miles of pipe, for the year 1929 the unaccounted for water was 21.9 per cent.; for 1930, 19.9 per cent.; for 1931, 20.6 per cent.; for 1932, 19.7 per cent.; for 1933, 19.2 per cent.

Henry T. Gidley.‡ I would like to ask Mr. Ross if the master meter was tested.

Mr. Ross. It was. During the course of the survey the Venturi meter was tested and found to be registering correctly.

Mr. Gidley. The reason I asked is because I had a large amount of unaccounted-for water, and I found the meter was 15 per cent. fast.

*Superintendent, Holyoke, Mass. †Superintendent, Milton, Mass.
‡Fairhaven Water Co., Fairhaven, Mass.

A RESUMÉ OF CHANGES IN PRACTICE AND RECENT IMPROVEMENTS IN WATER SUPPLY AT NEWTON, MASS.

BY RICHARD H. ELLIS.*

[*Read January 10, 1934.*]

A subject such as a resumé of a water department's activities over a period of some three and a half years, savors of annual reports which, while they might contain valuable and interesting data, would hardly be proper subjects for presentation before this ASSOCIATION. The recent work in Newton has however been somewhat unusual, and the author has attempted in the following presentation to cover the subject matter in a non-technical manner and to touch on various phases of management, design and practice so that there should be few who will not find something of interest.

The City. The city of Newton, having a population of approximately 70 000, is composed of ten almost independent communities including, Newton Corner, Newtonville, West Newton, Auburndale, Waban, Newton Highlands, Newton Upper Falls, Newton Lower Falls, Newton Centre and Chestnut Hill. Each of these communities has its own business center and one or more railroad stations, thirteen in all. It is interesting to note that there are ten different post offices located throughout the city.

The city as a whole contains some eighteen square miles of territory, the geographical center of which is but seven and nine-tenths miles from the State House in Boston. Located as it is, close to Boston, and being fortunate in having excellent train service, Newton has developed into a city populated largely by commuters. There are but few industries within the city, and the present zoning laws preclude the probability of much further industrial development—92 per cent. of the city being restricted to residential properties.

The growth of the city has been consistently rapid since 1880, the ten year increase varying from a low of 15.7 per cent. in the period 1910–1920 to a high of 41.8 per cent. during the halcyon period of 1920–1930. This rapid increase in population, with its accompanying demands, during the latter period has imposed upon the Newton Water Department, which in 1920 was already taxed to about the limit, some serious handicaps.

Source of Supply. Newton's water-works system was established in 1875. Ground water is obtained within a 677½ acre reservation situated south of the Charles River. The water obtained in this reservation flows at present by gravity through a 24-in. cast-iron pipe under the Charles River to a pump well located in the main pumping station on the Newton side of the Charles River.

*Water Commissioner, Newton, Mass.

In order to logically present the subject matter of this paper, the author desires to divide his discourse into two parts: (1) the result of a preliminary survey of the conditions which served as handicaps to proper and efficient management, a survey of the numerous complaints received, the reasons therefore and an outline of the corrective measures adopted; and (2) the development of a four-year program for the rehabilitation of the department.

Preliminary Survey of Conditions. The author's first impression upon becoming associated with the government of the city of Newton was that the shop routine and general office procedure were handicaps to efficient management. Subsequent investigation revealed that the primary reasons for this condition were as follows:

1. The working organization was without proper quarters.

A. There was no properly organized stock room nor was any attempt made to keep stock records.

(*a*) The only record of material used on a job was made up by the foreman and reported to the office for record and billing purposes on time reports.

(*b*) No check system was used to prevent the depletion of stock required.

B. Meter-repair work consisted merely of replacement of worn parts, and no systematic program for the inspection of meters was followed.

(*a*) Meters were removed and repaired only after they had been reported as stuck.

(*b*) Quarters were dark, crowded, and cluttered due to the fact that work of this nature was carried on in the same room with all other shop operations.

(*c*) No records were kept of meter repairs or tests.

2. The personnel was not properly trained to accomplish its work efficiently nor to properly render those services which the public has a right to expect from an efficient organization.

3. The office procedure (particularly as it applied to the issuance of rate bills) was inadequate for a community of approximately 70 000.

A. Severe peak loads were imposed on the general office and on the inspection force as a result of the practice of issuing simultaneously 15 500 water bills on January 1st and approximately 6 000 on July 1st, on an annual basis.

B. As a result of these peak loads, the public was disgruntled over the failure of the department to promptly report the result of the investigations of complaints, etc.

4. A serious situation existed which resulted in an almost infinite number of complaints of discolored and impalatable water.

5. The unit rate at which water was sold was unjustifiably high.

In order to undertake any work of a major nature it was evident that a properly laid out and well equipped shop was essential, and the government was first requested to grant authority to make studies which would lead to the development of such a structure.

The Shop. Studies were made to determine whether or not a new location would be more advantageous than that of the former shop, located

in Newtonville. Certain city-owned tracts in Newton Highlands would have brought the shop nearer to that territory which in all probability will see in the next twenty years the major development in Newton, but would have necessitated an average additional mileage of about two and a half miles and a consequent delay in responding to the more numerous emergency calls which normally would come from the more thickly and earlier developed sections of Newton, Newtonville, West Newton and

FIG. 1.—SHOP STRUCTURE BEFORE REMODELLING.

Auburndale. Preliminary sketches indicated that the development of a new structure of adequate proportions to provide for expansion would have cost in the neighborhood of $100 000.

The old shop had been built in 1892 as a combined shop and stable (Fig. 1). It was located at the intersection of two streets — Watertown and Edinboro. The latter was some eight feet higher in elevation than the former, thus providing two grade entrances, one to the shop and one to the stable. A third floor had, in the days of horse-drawn transportation, been used as a hay loft. From time to time a number of sheds had been built in the yard to house stock etc. and later to provide shelter for the automotive equipment.

The original studies indicated that it would be desirable to provide a total of 16 000 sq. ft. of floor space for the contemplated structure. In as much as there appeared to be as many disadvantages to any available new location as to that occupied by the existing shop, our attention was turned to the possibility of making some major alterations and additions

to it. A plan was developed which would make available 8 400 sq. ft. of floor space in the existing structure, and an addition of a block of 18 garage stalls, eight of which were assigned to dead storage, gave us an

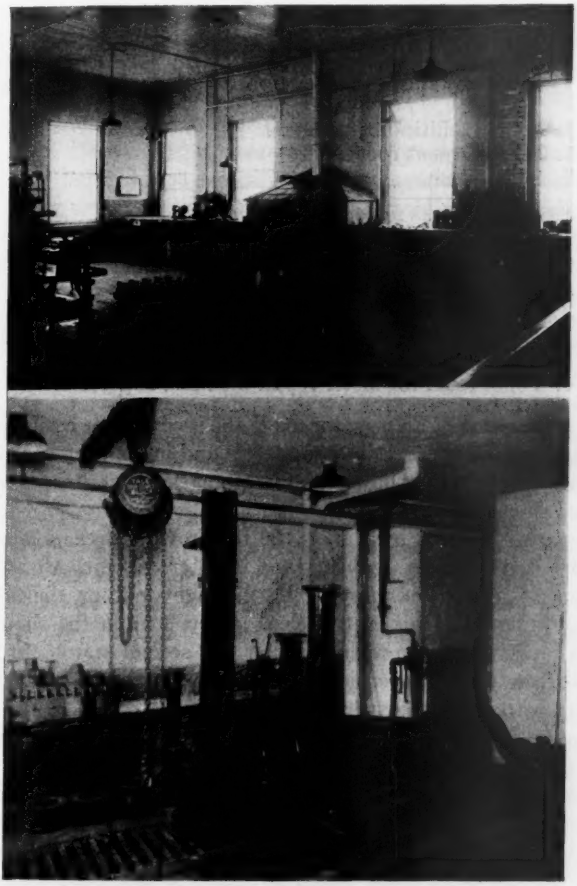

FIG. 2. ABOVE.—NEW METER REPAIR ROOM.
FIG. 3. BELOW.—NEW HYDRAULIC LABORATORY.

additional 4 700 sq. ft. making a total of 13 100. Due to the type of construction and the location selected for this latter structure considerably more room is available for additions to it as the growth of the department may require. The total cost of the remodelled building and the additions

represented an outlay of but $40 000—a net saving of $60 000 from the original plan for a new structure.

In the remodelled structure we were able to develop on the first floor:

1. Pipe shop, equipped with cutting and threading machinery (Fig. 2).
2. Laborer's room.
3. Locker room.
4. Hydraulic laboratory, used primarily for testing meters 2-in. or larger. This room is equipped with chain falls on track, etc. (Fig. 3).
5. Toilet facilities for laborers.
6. Disbursement room.
7. Boiler room.

The second floor was remodelled to accommodate:

1. Foreman's office.
2. Office and stock room to contain all small service stock, meter parts, etc.
3. Roomy meter repair and test room laid out to handle 4 000 to 8 000 meters per year and permit a five-year inspection program, three men being steadily employed.
4. Toilet room for meter-repair men and supervisors.

The installation of an elevator made the second floor easily accessible from either Watertown or Edinboro Streets and in addition permitted the development of the former hay loft into a storage room for tools and storage of unusual stock.

The garage as built is 90 ft. by 26 ft., provides accommodations for 10 vehicles and contains a wash stand and grease pit. All automobile maintenance is conducted at night by the night man on the first watch who in addition rings his watch clock every hour in the shop. Janitor service is rendered by the night man on the second watch. In this way all our cars are regularly serviced and the building is maintained in first class order without extra expense.

Shop Records. Upon completion of the shop it became possible to develop a logical shop practice. Proper stock records were installed providing a perpetual inventory so designed as to show the maximum and minimum amount of each item of stock it is permissible to carry, a coördinating requisition system whereby material required is requisitioned from the general office from which all material orders are issued. A job requisition system was installed whereby material required for work in progress is drawn from the stock room on a requisition bearing a job number. This number being the same as that appearing on the foreman's time record, the requisition sheet after having been entered by the stock clerk on his perpetual inventory is forwarded to the general office where it is filed until after the weekly payrolls are computed and checked. These payroll forms are so arranged as to set forth in addition to the aggregate time each employee works, the break-up of his work on any particular

part. The checking of the payrolls necessitates the calculation of the total value of all the work within the week on each and every job. The requisitions, each bearing the same job numbers as the payroll columns, are then readily combined with the labor tally and without being copied are transferred by the billing clerk directly to the bills and commitments or to the proper construction accounts.

Meter Repairs. The new meter shop provided by the remodelling of this plant was designed to accommodate the shop inspection of from 4 000 to 8 000 meters per year without further addition. This shop is equipped with a large drain tank into which all meters are placed on being brought into the shop. Here all exteriors are washed before the meters are removed to the repair bench which is equipped with upright drills, buffers, compressed air for drying and a sink with hot and cold water attachments for washing interiors. Acid tanks for thoroughly cleaning brass parts are conveniently located in a hooded and vented compartment accessible to all the repairmen. Testing apparatus and storage facilities are so placed as to reduce to a minimum the shop travel. A lathe is provided for necessary machinery of parts.

All stock used in the meter room is requisitioned from the stock room on meter job slips which are stamped at the beginning and end of the repair operation. The total repair cost—labor and parts—is calculated by the stock clerk, and a complete accounting of it is entered on a permanent meter-history card.

Meter Records. The meter records kept consist of an index by street and number of all services in the city, showing each meter set on each service, the date of setting, date of removal and the replacement. The meter-history card before referred to is indexed by make and number and shows where the meter has been set, dates of setting and removal, the reason therefore, the result of the test when removed, a notation of the condition of the meter, necessary repairs and their cost and the result of the final test. The city, by owning all the meters, is enabled when removing meters for the purpose of inspection to replace them at the time of removal. A light truck mounted with a body designed especially for the service is used for this purpose.

Within the last two and three-quarter years, the department has removed, reconditioned and repaired 12 300 meters. The result of this work so far accomplished would seem to indicate that the net revenue gain will approximate 18 per cent., an amount equal to nearly 4.5 times the cost of the inspection work.

A rather unlooked for result of the furnishing of good working quarters was the increased moral of the working organization. The change of environment, the facilities available and the definite assignment of responsibilities together with the segregation of the several operations performed definitely increased the efficiency of the organization. The men were trained in the proper use and maintenance of the tools and equipment.

Definite efforts were made to offer advisory service based on actual surveys of conditions to those who reported a shortage of water in their house piping. Service records are being checked over and dead services disconnected at the main. A continuous hydrant inspection program has been operating for two years and a gate inspection program is to be started this spring.

Former System of Reading and Billing. The former system of reading and billing required that the meter reader or inspector should complete the reading of one complete ward before turning in his books. These ward books, of which there were seven, contained on the average 2 100 accounts each. The accounts, after being read were computed in the office, the computations entered in a ledger book from which the bills and commitments were copied. This procedure required the making of three copies, one by hand and two typewritten. All services were charged an annual minimum advance rate which was billed, together with any excess over the previous annual minimum as of January 1st. All meters were read semi-annually, and if water to the value of more than the minimum had been consumed by that time an "excess" bill was rendered as of July 1st. All "excess" water not so billed was charged on the subsequent annual bill rendered the first of the following year.

The system was confusing. It created two severe peak loads per year on the office organization, required the employment of temporary help and overwhelmed the inspection force following each billing period with such a volume of work that many of the inquiries resultant from the receipt of unusually high bills could not be promptly investigated. As a result the report lost much of its value due to the change in conditions which might meanwhile have occurred and the long period required to read all the meters before the billing date, and the public was unable to coördinate its use of water with the bills rendered.

Careful study of the possibilities of developing a more satisfactory system of reading, billing and inspection revealed the fact that the two regular readers, properly routed, and a supervising inspector who should investigate the complaints, pick up missed readings and untangle any irregularities reported, would be ample to handle the routine work involved in reading all the 15 500 meters quarterly.

New Reading and Billing Procedure. The city was divided into three districts and a stagger system developed whereby the meters in each of the three districts are now read four times a year at intervals of three months and in such a manner that no two districts are read within the same month. Modern billing machinery was purchased and a "bookless bookkeeping" type of billing procedure was adopted. All accounts are figured in the office and are entered in the original meter-readers' books, each of which contains on the average of 290 accounts or two days work. The billing operation is carried on simultaneously with the reading; the operator, after having prepared her bills and commitments on addressing equipment, fabricates at one operation both the bill and the commitment. Upon completion of the

billing operation the commitments, having been automatically footed to show the totals of the present and past readings, the consumption and the amount of the bills, are checked to eliminate the only possible chance for personal error — the calculation of the rate. Approximately 5 000 bills are thus rendered at the first of every month.

The whole billing process is a straightforward operation having no duplication of effort and requires no copying which would necessitate back-reading or a tedious checking operation and requires little more than one clerk's entire time.

The effect of the system as adopted was to entirely eliminate the peak loads on both the office and inspection forces, equalizing each month's work and thus enabling the department to render prompt service on the investigation of complaints which were greatly reduced due to the uniformity of the interval between readings. In general the public appreciated the receipt of a bill four times a year instead of at longer intervals, and the fact that full data as to the detail of the bill — including the date of reading, the present and past reading, the amount of the consumption recorded, all of which did not formerly appear, — was pleasing to the consumers. One other result that is of importance to the operation of the department is that the clerk, in calculating the consumption, reports daily to the meter department any meters which are failing to register. These meters are usually replaced by reconditioned meters within a period of three days.

Quality of Water. Throughout the entire year, but particularly during the summer months of greatest consumption, complaints in large numbers were received of discolored or unpalatable water. It was a known fact that the water as obtained at the source of supply is rather high in CO_2 and of a low pH value and therefore quite highly corrosive. Plans had been developed to conduct a rather large-scale experiment with several different treatment methods in order to determine which method as most suitable for use with Newton water.

Experiment on Corrosion Control. About the beginning of 1931, this experimental set-up was completed and put into operation, and by the end of the same year the department, basing its action on the experimental results, began a corrosion control treatment which we believe has been highly successful and has been largely responsible for the greatly improved condition of the water as drawn from the mains. Action on house piping has we believe been greatly minimized, and a direct saving thus should accrue to the house owner.

The details of this experiment and the treatment of the water being the subject matter of another paper to appear in this Journal, the author will trespass no further on the subject.

Dead Ends. An analysis of the many complaints received prior to 1930 indicated that the majority arose in districts which were fed by "dead ended" mains. Due to two primary reasons, the city of Newton had much more than its share of such mains. In all probability due to the demand

for main extensions during the ten-year period of maximum growth, 1920–1930, the distribution crews were unable to do much more than the minimum amount of work necessary to make water available to the new development which had sprung up. Thus few through connections were made during this period.

Another factor which the author believes contributed to the creation of the serious "dead ended" condition was the rigid pursuance of the practice of insisting on the guarantee of a proper financial return to the department for every dollar invested in main-pipe construction. Such a practice, while it has much merit, can be carried too far. Newton had proceeded along these lines to such an extreme that large portions of the distribution system suffered from lack of circulation and many large developments had inadequate fire protection. It was evident that for the good of the system it was inexpedient to follow this practice rigidly except in the case of new developments where the department insists on a satisfactory layout.

Appropriations were granted to start on a program of elimination of "dead ended mains." This work has been continually under way since the latter part of 1930. Great satisfaction has been expressed by residents of districts where this work has been carried to conclusion. As a result of the intensive work done during 1930 and 1931 in accomplishing or getting under way the work just outlined, the confusion which had previously precluded the possibility of logical and intensive study of some of the larger essential problems disappeared, and by the end of 1931 the department was able to develop a definite four-year program the completion of which has already yielded and is expected to further furnish very beneficial results.

Four-Year Program. During the latter part of the year 1931 a survey of the remaining most pertinent requirements of the department led to the formulation of a definite program the practical accomplishment of which is expected by the end of 1935. This program provided for the following improvements:

1. The elimination of "dead ended" conditions throughout the city by the construction of some 300 extensions having a total length of 18 miles at an estimated cost of $300 000. To date this project has served chiefly to take up any slack which might otherwise be encountered by the distribution crew. Of the work proposed about 25 per cent. has been completed to date.

2. The elimination of 6⅓ miles of 4-in. mains at an estimated cost of $100 000. No work under this project has been undertaken except in cases where highways have been resurfaced.

3. The reconstruction of 5 miles of distribution mains in Boylston Street in connection with the reconstruction of the Boston and Worcester turnpike, providing for the installation of a 16-in. trunk main on the north side of this highway and 6-in. and 8-in. distribution mains on the south side of the highway at an estimated cost of $92 000. This work has been completed within the estimated figure.

4. The development of the source of supply to enable the department to obtain the maximum amount of water available under the statutory limitations.

5. The remodelling of the pumping station and the installation of new pump equipment of greater capacity.

Development of Source. In connection with the fourth of these recommendations — the development of the source — the author desires to review briefly its development up to 1926 and then to describe the work done to make available the amount of water required.

Newton, as before mentioned, obtains its supply from 677½ acres of water-bearing gravel lands in Needham. Originally water was obtained by infiltration from the adjacent Charles River into a gravel basin through the center of which was constructed at a depth of 10 ft. below the surface a 4 ft. by 4 ft. wooden collecting conduit. From this conduit — 1 575 ft. in length, — the water was discharged by gravity through a 24-in. cast-iron pipe, under the Charles River to the pump well in the pumping station.

The yield from this infiltration basin proving inadequate within a short time and the depth of the filtering medium being insufficient, it was decided to continue the conduit 2 194 ft.—giving a total length of 3 769 ft. The conduit so constructed was fed by 209 2½-in. driven wells, the yield from which was so depleted by 1931 as to amount to a negligible quantity. The conduit also served to intercept the natural ground water flowing toward the Charles River.

As the demand for water increased, a dug well, known as Well No. 1, was constructed in 1911. This well, 28 ft. in diameter and 38 ft. deep is equipped with duplicate, vertical, motor-driven, submerged, centrifugal pumping units which discharge through a cast-iron main into the upper end of the wooden conduit.

Dug Well No. 2, installed in 1926 to further augment the supply, is 33 ft. in diameter and 42 ft. in depth and is equipped in a like manner to Well No. 1. This second well also discharges through a cast-iron main into the wooden conduit. The natural ground water supply to these wells is augmented as required by water from the Charles River pumped into three flooding or charging basins located in gravel hills west of the wells. The yield of the source so developed was 5 m.g.d. which was the maximum amount that the pumping equipment in the main station was capable of handling. Any peak requirements of water were of necessity purchased from the Metropolitan District Commission of Boston.

There were apparently three alternative courses open to the city: (1) to abandon its source and pumping plant and to purchase its entire supply of water from the Metropolitan District Commission; (2) to enter into an agreement with the Metropolitan District Commission whereby the city might as a matter of regular practice continue to purchase all water required beyond its ability to pump; (3) to develop the yield of its source and to install in its main station equipment of sufficient capacity as to make available the 8 m.g.d. of water to which the city is entitled under State regulations.

The terms as quoted by the Metropolitan District Commission for the

city's full requirements would have involved the expenditure annually for water of an amount equal to two and one-half times the 1930 cost of supplying its own requirements. This extra cost amounting to approximately $120 000 per annum diverted our studies to the other two courses available.

The amount of the saving which would have resulted from the retention of the city's own supply works under the second scheme was dependent only on the economies which could be developed in its operation. It was figured that the least amount of savings that would thus be returned to the city, provided modern equipment was installed to replace the then existing pumping equipment, would completely justify the expenditures necessary. This conclusion therefore led to further studies as to the feasibility of further developing the source and of installing pumping equipment of greater capacity and resulted in the adoption of the third available course.

Negotiations were opened which led to the purchase of 23 acres of gravel land, 300 ft. in width on which were developed two additional infiltration bowls or charging basins which together with the experimental bowl have given quite satisfactory evidence that the desired result has been obtained.

The Pumping Station. The main pumping station located on Needham Street in the Upper Falls was equipped with a 5-m.g.d. Worthington duplex compound unit installed in 1875 and a 5-m.g.d. Barr vertical cross compound pump installed in 1904. The boiler equipment consisted of two 150-lb. pressure 300, h.p. horizontal tubular Belpair boilers each of which was 42 years old.

In contemplating the installation of equipment of sufficient capacity to supply the needs of the city it was evident that there was little of value in the existing equipment. Studies were made of different types of machinery, including motor driven, Diesel engine and steam turbine equipment to determine that which was best adapted to Newton's requirements. Inasmuch as the use of several electrical units in the Water-Works Reservation required the operation of electric motors totalling about 125 h.p. the feasibility of installing generating equipment was considered on the two latter types—Diesel and steam turbine equipment. Since it had been found decidedly more economical to utilize the old structure than to build a new one the installation of Diesel equipment of acceptable design was more or less out of the question due to the space requirements and foundation difficulties.

Specifications were prepared and proposals and performance guarantees received on both motor-driven and steam-turbine-driven units with generators, and of 4-, 6- and 8-m.g.d. capacity. With the evidence thus secured as to the capital investment required and as to the operating costs under both schemes it was possible to evaluate the advantages of one type over the other. This procedure led to the selection of steam equipment and to the award of the contract for it, based on the cost and on the efficiency guarantees. Proposals were then received for the work involved in remodel-

ling the old structure, installation of two 250-lb. pressure, 250-h.p. water-tube boilers and auxiliary equipment.

Work was started on the remodelling of the station in September, 1932 and continued steadily until completion in August of this past year.

In connection with the accomplishment of this construction it is of interest to note that the city of Newton in generating the energy consumed in the Water-Works Reservation has reduced the cost of power from 9.9 to 4.15 mills per k.w.h., and the fuel cost for pumping water from $6.28 to the low of $3.75 per m.g. against a head of 274 ft., a net of $1.373 per m.g. per 100-ft. lift.

The total cost of developing the water requirements of the city at the source and of delivering the water into the mains has been reduced by at least $17 000, not taking into consideration the saving to be expected from our increased capacity. The amount thus saved compounded at 4 per cent. would depreciate the entire equipment within 10 years, the total cost of the installation being $199 350.

Results. As a tangible result of the whole work undertaken, the department, upon developing estimates for the work outlined in the four-year program previously referred to, and taking into consideration the increase in revenue which apparently was to accrue as a result of the meter-inspection work, was enabled to announce as of January 1, 1932 a rate reduction of 10 per cent. — from 25c to 22½c per hundred cubic feet. The economies subsequently effected, particularly as a result of the increased efficiency of the meters and the reduced pumping and power costs, permitted again on January 1, 1934 a further reduction of 11.1 per cent. from 22½c to 20c per 100 cubic feet as a base rate.

The entire work as outlined involving the expenditure to date of $468 000, with the exception of monies secured from the sale of $200 000 worth of bonds for extension purposes, has been accomplished from current revenue. The cash surplus in turn has been increased since 1929 from a deficit of $23 600 to a net surplus of $170 000.

In presenting this resumé of changes in practice and a review of the recent improvements in the Newton water system, recognition should be given to His Honor Mayor Weeks, and the members of the Aldermanic Boards of 1930, 1931, 1932 and 1933 who by their whole-hearted support have made these accomplishments possible.

ENGINEERING PROBLEMS CONNECTED WITH RECENT IMPROVEMENTS TO THE NEWTON, MASS., WATER SUPPLYING WORKS.

BY GEORGE A. SAMPSON.*

[*Read January 10, 1934.*]

The purpose of this paper is to present some of the engineering problems connected with the recent extensive improvements to the water supplying works of the City of Newton, Mass.

Newton is a residential community adjoining the city of Boston. Owing to its geographical location, coupled with a well conceived civic development and many other advantages, the city of Newton, especially in the last two decades, has enjoyed a marked increase in population and water consumption, as recorded in Table 1.

TABLE 1.—POPULATION AND WATER CONSUMPTION.

Year	Population	Water Consumption	
		Gallons per day	Gallons per Capita per day
1895	27 600	1 801 200	64.8
1900	33 587	2 086 400	62.1
1905	37 300	2 151 300	57.7
1910	39 806	2 505 400	62 5
1915	43 800	2 830 500	64.6
1920	46 054	3 699 400	79.6
1925	53 003	4 137 500	78.1
1930	65 276	5 002 700	76.6

IMPROVEMENTS IN SOURCES OF SUPPLY.

Existing Supply Sources. The supply is a ground water obtained from an unusual gravel deposit along the Charles River in the town of Needham, the extent of the present sources being shown on Figure 1.

The original works were constructed in 1876 and were extended periodically to keep pace with the ever increasing water consumption. By the year 1920 a water consumption of 3 699 400 g.p.d. had again about equalled the capacity of the supplying works. At that time the collecting system consisted of the following parts:

(1). A 4 ft. by 4 ft. wooden infiltration conduit 3 769 ft. in length installed in 1876 and extended in 1889. Connected to this conduit are 209 2½-in. tubular wells driven into the gravel strata. If the yield from these wells was ever a substantial quantity of water, it is now negligible.

(2). A double 24-in. vitrified tile infiltration conduit 3 188 ft. in length, constructed in 1894 along the margin of a cat-tail swamp and discharging by gravity into a junction manhole at the upper end of the wooden conduit just south of Kendrick Street.

*Of Weston & Sampson, Consulting Engineers, Boston, Mass.

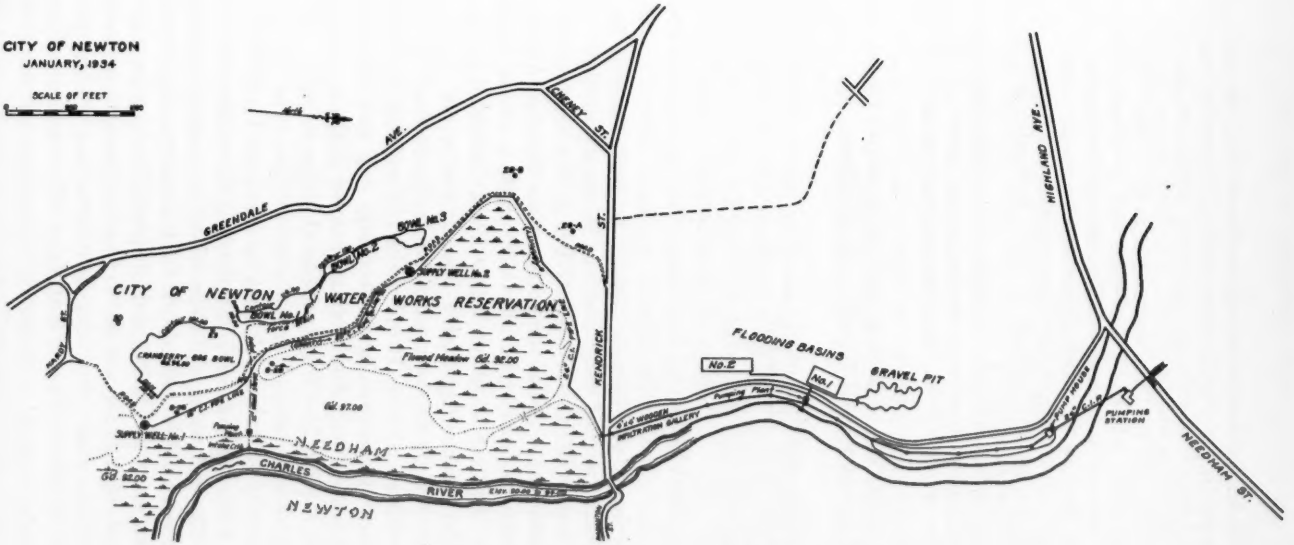

FIG. 1—SOURCES OF WATER SUPPLY, NEWTON, MASS.

(3). A 30-in. concrete collecting conduit 1 425 ft. in length, fed by a considerable number of 2½-in. driven wells. This conduit was laid in 1908 and 1909 and connected with the previously mentioned junction manhole.

(4). A large dug well (Well No. 1) of 28-ft. inside diameter, 38 ft. in depth, constructed in 1911 at a contract price of $8 990. This well is a concrete cylinder sunk into the gravel layer and is covered with a concrete dome roof. The well is located 3 595 ft. beyond the junction manhole and 500 ft. from the Charles River. Two vertical, motor driven, submerged centrifugal pumping units, each with a capacity of 1.5 m.g.d., discharging through a 24-in. vitrified tile pipe, 2 170 ft. in length, into the upper end of the 30-in. concrete conduit.

(5). A 24-in. cast-iron gravity supply main, 750 ft. in length, extending beneath the Charles River and connecting the lower end of the wooden conduit with a pump well at the main pumping station in Newton Upper Falls.

The average yield of these ground water sources for the period from 1920 to 1925 inclusive was 3 775 000 g.p.d. as follows:

	Gal. per day
Wooden conduit	2 000 000
Double 24-in. tile conduit	750 000
30-in. Concrete conduit	225 000
Dug Well No. 1	800 000
Total Yield	3 775 000

Changes in and Additions to Supply. In 1921, the city, for the first time, purchased 5 m.g. of water from the Metropolitan Water Works and in 1925 the water purchased reached 176 m.g. The water supply problem was then studied by a special committee of the Board of Aldermen of the city of Newton, assisted by Mr. J. C. Whitney, a former Secretary and President of the New England Water Works Association, then Water Commissioner; by the Engineering Department of the city of Newton; by the Massachusetts Department of Public Health, and by the writer's firm as consulting engineers. It was decided to augment the ground water sources at once by constructing another large dug well (Well No. 2) at a location that was selected after driving a number of 2½-in. test wells.

Well No. 2. Well No. 2 was completed in the spring of 1927 and the pumping equipment installed in the early summer. The well is 33 ft. in inside diameter by 42 ft. in depth, sunk into the gravel at the foot of a large upland water-bearing area and just above the cat-tail swamp previously mentioned. The well casing is of 16-in. reinforced concrete, perforated with 60 4-in. diameter holes in three rows in the lower 12 ft. After the excavation had progressed to a depth of 8 ft. and into ground water a concrete cylindrical ring with a cutting edge was cast in place and sunk by excavating from within. Successive rings were then constructed and the excavation progressed until the desired depth was attained. The contractor was equipped for removing the material by pumping, but owing to the size of the stones encountered in the gravel this method had to be abandoned. A clam shell

bucket was then used and the excavation substantially unwatered during the progress of the work. Although some difficulty was experienced in sinking the cylinder vertically, this was overcome by digging on the high side and bracing against the outside of the shell. Real trouble was encountered, however, when the outside gravel began to slide under the bottom of the casing which developed into cave-ins 20 ft. wide at the top. As a result, the excavation removed was at least twice the contents of the cylinder and a large amount of backfill was required.

The contract price for constructing the well, including a concrete roof and small brick pump house, was $11 400. This figure does not represent the actual cost to the contractor, nor is it more than one-half of a fair average price for a well of this type and size.

The pumping equipment consists of two, duplicate, vertical, motor driven, submerged, centrifugal pumps, each designed for a capacity of 2 772 000 g.p.d. The contract price installed, including the electrical work, was $4 646.

The natural yield of Well No. 2 was found to be 1 160 000 g.p.d.

Sources Abandoned. The results of a sanitary survey in 1926 showed that the double 24-in. tile infiltration conduit and the 30-in. concrete collecting conduit were so located as to be affected by high water in the Charles River and that at such times these sources furnished a supply of inferior quality. It was, therefore, thought advisable to abandon these two conduits which furnished 975 000 g.p.d. When this decision was reached it introduced the problem of conveying the discharge from Well No. 1 and from the proposed Well No. 2 to the wooden conduit. It was decided to install a cast-iron main skirting the swamp, consisting of 1 310 ft. of 16-in. and 1 315 ft. of 20-in. pipe from Well No. 1 to Well No. 2 and 3 185 ft. of 24-in. pipe from Well No. 2 to the wooden conduit. The 16-in. size was used to provide for a possible future dug well south of Well No. 1 and the increase to 20 in. was decided upon to serve another possible well midway between Wells No. 1 and No. 2.

The completion of Well No. 2 and the abandonment of the tile and concrete conduits resulted in a net increase in yield of only 185 000 g.p.d., to a total of 3 960 000 g.p.d. as compared with an average daily water consumption for the year 1927 of 4 420 000 gal.

Flooding Basins. The next step in the development was an experiment to determine the practicability of increasing the yield of Wells No. 1 and No. 2 by pumping water from the Charles River into a natural basin in the gravel hills situated between and above the two wells in the location of Bowl No. 1, as shown in Figure 1. This experiment is of especial interest as it marked the beginning of a flooding development to augment the ground water supply which is rather unusual in this country.

An area in the form of a cross with a width of 40 ft. and a total length of 700 ft. was cleared of scrub oaks and stripped to the gravel. In the meantime a temporary electric motor driven centrifugal pump with a capacity

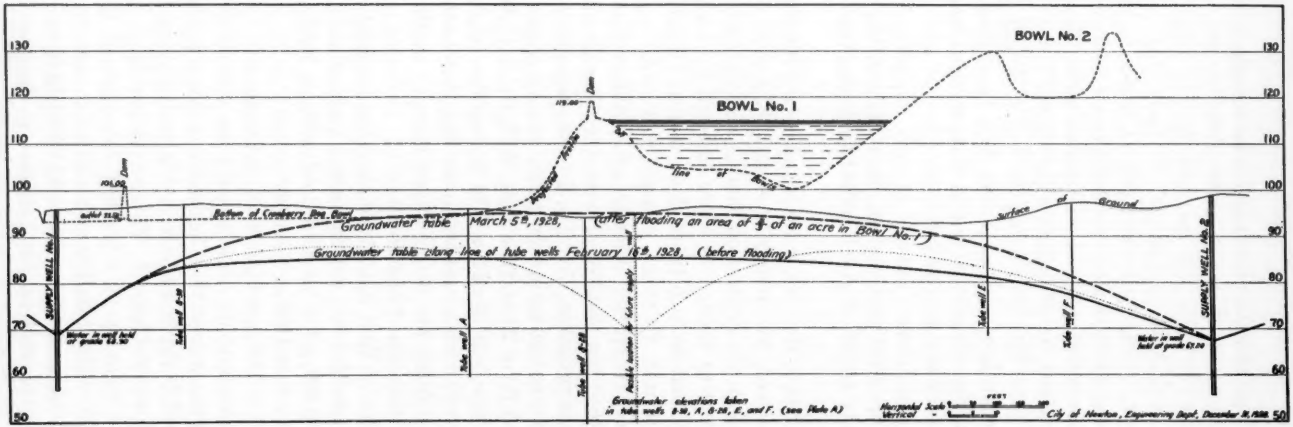

FIG. 2—EFFECT OF FLOODING BOWL NO. 1 ON GROUND WATER TABLE.

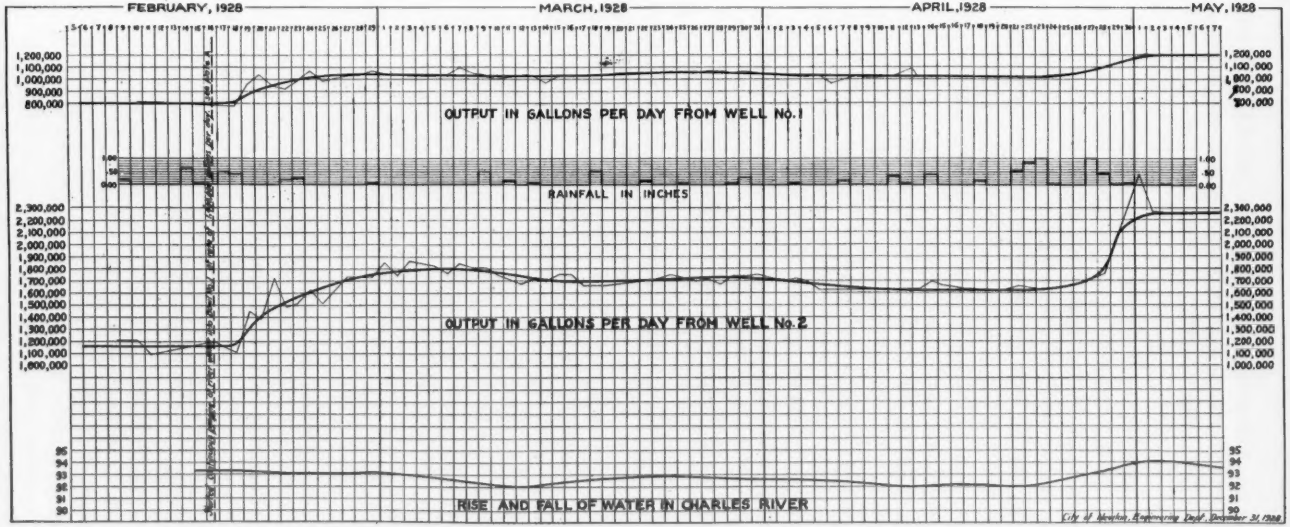
FEBRUARY, 1928
MARCH, 1928
APRIL, 1928
MAY, 1928
OUTPUT IN GALLONS PER DAY FROM WELL No. 1
RAINFALL IN INCHES
OUTPUT IN GALLONS PER DAY FROM WELL No. 2
RISE AND FALL OF WATER IN CHARLES RIVER

FIG. 3—EFFECT OF FLOODING BOWL NO. 1 ON YIELD OF WELLS.

of 1 400 000 g.p.d. was erected near the river and a 12-in. wood pipe force main laid from the pump to the basin. Continuous flooding began on February 16, 1928. After two days of pumping a rise of several inches was noted in the elevation of the water in several 2½-in. test wells driven along the margin of the swamp. At the same time there was a slight increase in the yield of Well No. 1 and Well No. 2, particularly No. 2. On March 5 the yield of Well No. 1 had increased from 800 000 to 1 030 000 g.p.d. and Well No. 2 from 1 160 000 to 1 800 000 g.p.d. or a total increase of 870 000 g.p.d., and the ground water between the wells had risen several feet as indicated in Figures 2 and 3. From a flooding area of 0.64 acre, 870 000 g.p.d. or 62 per cent. of the water pumped from the river had been collected in the wells about one-quarter mile distant.

Before and during the flooding, numerous samples of water were collected from various 2½-in. test wells located at different distances from the basin, and from Wells No. 1 and No. 2, in order to determine the radius of influence of flooding on the quality of the ground water. In general it was found that a lateral distance of 150 ft. from the flooding basin to the point of collection was more than ample, in the rather coarse gravel strata, for the complete purification of the river water. This distance was established as a minimum in the design of subsequent flooding basins.

Following this successful experiment the entire area of Bowl No. 1 amounting to 1.8 acres as well as another nearby natural basin at a higher elevation—Bowl No. 2 with an area of 0.65 acre—were stripped to the surface of the gravel during the fall of 1928. A permanent intake crib and a pumping station equipped with a 3 500 000-g.p.d. electric motor driven centrifugal pump were constructed at the river and the 12-in. wood pipe line was extended to Bowl No. 2.

The new flooding system was placed in operation early in 1929 and has been used since that time for augmenting the yields of Wells No. 1 and No. 2. Later in the year 1929 a third basin (Bowl No. 3) located directly above Well No. 2 and 150 ft. distant was constructed to serve as a more direct supplement to this well. With the three bowls the yields of Wells No. 1 and No. 2 were 1 030 000 and 2 300 000 g.p.d. respectively, or a total of 3 330 000 g.p.d. which is an increase of 1 370 000 g.p.d. over the natural yield. The amount of water pumped from the river is controlled by throttling the discharge valve at the pump and is varied in accordance with the need for flooding. To recover 1 370 000 g.p.d. requires about 2 500 000 g.p.d. of flooding water. The total yield of the ground water sources was now 5 330 000 g.p.d. but still inadequate for periods of only slightly greater than average consumption.

Additional Flooding. The next step in developing the ground water supply was another experiment in flooding in the early part of 1930. A gravel pit located about midway of the wooden conduit as shown in Figure 1 had just been abandoned and it seemed logical to flood this area. The main advantage in this method of flooding over the first method is found in a

single pumping. The water is collected directly in the wooden conduit instead of in wells and one pumping is eliminated.

After a temporary pumping station had been erected at the river and a force main installed, flooding began in May 1930. Within two days there was an increase in the water level in the wooden conduit of 3 in. About 1 000 000 g.p.d. were then discharged continuously into the gravel pit for several weeks, of which 750 000 g.p.d. were intercepted by the wooden conduit. The minimum lateral distance from the pit to the conduit was 150 ft., and here again the sanitary protection was ample. The results of this flooding were even more satisfactory than for Wells No. 1 and No. 2.

The total yield was now 6 080 000 g.p.d., and it seemed quite certain that the sources could be developed to the full allowable legal limit of 8 000 000 g.p.d. by constructing two or more artificial flooding basins on the gravel area above the wooden conduit between the experimental gravel pit and Kendrick St. The legislative regulations under which the city of Newton obtains water from the Charles River and the adjoining water works reservation as finally enacted in 1913 and approved by the Massachusetts Department of Public Health are as follows:

8 000 000 g.p.d. at all times between December 5 and June 5.
8 000 000 g.p.d. from the 5th of any month to the 5th of the following month from June 5 to December 5 when the rainfall for the preceding six calendar months as determined by averaging the rainfalls at several nearby prescribed gaging stations exceeds 19 in.
7 000 000 g.p.d. when the rainfall is less than 19 in. but exceeds 18 in.
6 000 000 g.p.d. when the rainfall is less than 18 in. but exceeds 17 in.
5 000 000 g.p.d. when the rainfall is less than 17 in.

After these developments the amount of water that could be supplied to the city was limited not so much by the source as by the pumping equipment at the main station across the river. The main operating unit was a Barr, vertical, crank and fly-wheel, cross-compound steam pumping engine installed in 1904, limited to a maximum capacity of 5 200 000 g.p.d. but more judiciously operated at 4 500 000 g.p.d. A standby unit consisted of a 5-m.g.d. Worthington, horizontal, compound, duplex, double acting, steam pumping engine installed in 1875 and operated only as necessary. The real problem had now shifted from the source of supply to the pumping station.

At this stage Richard H. Ellis, President of the New England Water Works Association during the past year, was appointed Water Commissioner and with the assistance of the writer's firm as consulting engineers, began an intensive study of the pumping facilities. Following a decision to provide a pumping capacity of at least 8 000 000 g.p.d. the next and last step in the development of the sources to meet this demand was begun.

After considering additional dug wells it was decided to further flood the area above the wooden conduit. Two flooding basins were constructed as shown in Figure 1, having a total area of 1.7 acres and a permanent pump-

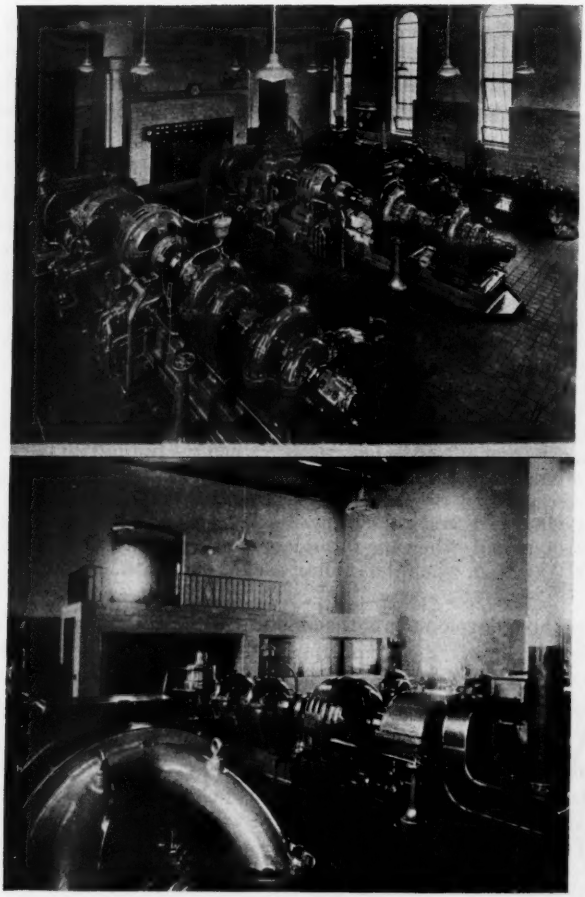

FIG. 4.—MAIN FLOOR OF REMODELLED PUMPING STATION

ing station for delivering water to the two basins and to the gravel pit. This flooding plant was placed in operation in the early summer of 1933 and has been in operation since that time as necessary to maintain a suitable ground water elevation in the wooden conduit. Although the yield of the sources as now developed has not been tested for a sufficient period it is believed that at least 8 000 000 g.p.d. are available, more than 50 per cent. of which are the result of flooding gravel areas.

Improvements in Pumping.

Pumping Station. The decision to install new pumping equipment of increased capacity was reached only after an extended comparison between developing the Newton supplying works and changing to the Metropolitan supply, in whole or in part. It was found that the city was not justified in shouldering the additional cost for Metropolitan water.

Early in 1932 preliminary comparisons of various types of pumping machinery were begun, including electric motors, steam turbines and Diesel oil engines as prime movers. On account of the increased first cost, additional space required, foundation difficulties and other related reasons connected with Diesel engines, the comparison was finally confined to electric motor and steam turbine driven centrifugal pumps, the latter units without and with attached generators for developing electric power for the motor driven pumps in the water works reservation, electric lighting, etc.

Although preliminary proposals for both electric and steam units seemed to favor steam power, it was decided to prepare specifications and to receive formal bids on both types of pumping equipment. Because the then existing steam plant consisted of two 300-h.p. Belpaire locomotive type boilers installed in 1890 with a maximum allowable steam pressure of 125 lb. and no superheat, bids were also invited on a new boiler plant.

Proposals were received on June 15, 1932 and after an exhaustive comparison of installation costs, fixed charges, operating costs and other factors the decision was reached to install steam turbine driven centrifugal pumps with generators and to replace the boiler plant, using pulverized coal for fuel.

Following this decision the contract for the pumping equipment was awarded to the Turbine Equipment Company of New England and for the boilers to the Erie City Iron works.

Pumping Equipment. There are three pumping units each consisting of a steam turbine, double helical reduction gears, alternating current generator, exciter and two pumps in series, as shown in Fig. 4. The generators and exciters were furnished by the Electric Machinery Mfg. Company. The De Laval Steam Turbine Company made the turbines, gears, pumps and cast iron bed plates and mounted the units.

The three generators are duplicates, each 200 k.w. at 80 per cent. power factor, 3 phase, 60 cycle, 4 000 volt, alternating current. The exciters are

mounted on the main shaft. The guaranteed efficiencies of the generators vary from 94.4 per cent. at full load to 92.0 per cent. at one-half load.

The principal characteristics of the units are given in Table 2.

TABLE 2—PUMPING UNITS.

Item	Unit No. 1	Unit No. 2	Unit No. 3
Number of Stages of Turbine	9	9	9
Speed of Turbine—r.p.m.	5 980	6 060	6 069
Speed of Generator and Pump—r.p.m.	1 200	1 200	1 200
Brake Horse Power of Turbine.	600	775	1 000
Capacity of Pump—g.p.m.	2 950	4 175	5 560
Dynamic Pumping Head—ft.	272	295	340
Capacity of Generator, k.v.a.	250	250	250
Guaranteed Efficiency of Pumps—per cent.	85	86	87
Steam Pressure at Turbine Throttle—lb. per sq. in.	230	230	230
Superheat at Turbine Throttle—°F.	125	125	125
Vacuum at Turbine Exhaust—in. Mercury	29	29	29
Guaranteed Duty—ft. lb. per 1000 lb. steam at 115 k.w. generator load	168 200 000	186 600 000	183 400 000
Guaranteed Water Rate—lb. per b.h.p. hr. at 115 k.w. generator load	10.6	9.7	10.0

Pump Well. The water from the sources flows by gravity to a pump well beneath the floor of the pump room which is at elevation 95.75. The capacity of the well is 20 000 gal. with the bottom at about elevation 82.25. The supply enters through a 24-in. butterfly valve equipped with a float to close the valve when the water reaches elevation 93.5. The water passes through a set of fine bronze wire screens (4 meshes per lineal inch), raised by a ½-ton electric hoist, and then into the suction well. The level of the water in the well is shown by an indicating float gage with a red hand set at the low-water danger point. In the screen chamber the water is treated with soda ash by means of a dry chemical feeder to neutralize its corrosive action and with liquid chlorine as a disinfecting agent.

Each pump has a separate suction ending in a bell-mouth at elevation 83.25. A water works type surface condenser is installed in each suction main beneath the pump room floor. Forced injection water from the city supply is also provided for each condenser until the pump is in operation or in case it is desired to run the generator without the pump. Each condenser is equipped with a 6-in. atmospheric relief valve discharging into the suction well.

Pumps. The pumps have a bottom suction and horizontal discharge with the series piping extending beneath the floor. The discharge from the second-stage pump is through the floor and through a horizontal swing check valve and hydraulically operated gate valve to a 24-in. force main outside of the pumping station connecting with the distribution system and a 10-m.g. covered concrete storage reservoir on Waban Hill having a flow line at elevation 320. The hydraulic cylinders of the valves are operated by

a lever handle 4-way cock mounted on a floor stand near the turbines. A 24-in. Venturi meter tube is installed in the 24-in. force main with an indicating, recording, registering instrument erected on the floor of the pump room near the switchboard.

The pumps are primed by a steam jet air pump, the main function of which is to remove the air from the condenser. The casings of the pumps are connected through an air valve to the first stage where a vacuum of 25 in. of mercury is maintained.

By substituting new impellers only, of slightly larger diameter, the pumps will deliver 5, 7 and 9 m.g.d.,respectively, against the corresponding pumping heads. The steam supply, turbines, gears, shafts and other details are designed for the increased horse powers even with the generators developing full load.

Condensate. An interesting feature of the installation is the method used for removing the condensate from the condensers by means of Schutte & Koerting water eductors submerged in the suction well. Each eductor is supplied by a 2-in. pressure line from the city main, the water first passing through the turbine oil cooler and steam jet air pump. The water used through the eductors returns to the suction well and amounts to 25, 33 and 40 g.p.m. for the three units, or about 0.75 per cent. of the discharge of the pumps.

Oiling System. An oil reservoir is provided in the base of each turbine. The oil pump connected with the turbine takes oil from this reservoir and delivers it through an oil cooler to one of three overhead storage tanks of 150-gal. capacity located on an elevated platform in the boiler room. From the overhead tank in use, the oil flows by gravity through a centrifugal purifier and is then delivered by a 1/6 h.p. motor driven rotary pump to the turbine gears and bearings through a sight glass and strainer. The excess oil pumped to the overhead tank and not used through the purifier is returned to the reservoir through an overflow pipe.

Each turbine is also equipped with an auxiliary oil pump for use in circulating the oil before the turbine pumps are in operation. No. 1 turbine is provided with a steam driven reciprocating oil pump located beneath the pump room floor. Turbines No. 2 and No. 3 are equipped with rotary oil pumps driven by a 1-h.p. motor and mounted on a steel bracket near the reduction gears. No. 1 unit can then be started even if electricity is not available.

Thermometers are mounted at the inlet and outlet ends of the oil cooler and on the oil feed line to the gears and bearings. A sight flow indicator is also inserted in the outlet line from the oil cooler.

Switchboard. The electrical switchboard was furnished by the Electric Machinery Mfg. Company and is of the "dead front" type with the bus and switch structures, transformers and other equipment located in a brick vault behind the board, access to which is through a door at one end.

In addition to the usual electric meters, switches, controls and syn-

chroscope the switchboard is also provided with a push button station for controlling the four pumping plants in the water-works reservation. There is a "start" and "stop" button for each plant and indicating lights as follows: When the "start" button is pushed the completion of the electric circuit is indicated on the switchboard by an amber light. As soon as the pump is delivering water this fact is shown by a green light which is actuated by a pressure switch connected to the discharge main near the pump. The distance from the switchboard to the auxiliary pumping stations varies from ½ to 2 miles.

Gage Board. The gage board and gages were furnished by the Ashton Valve Company. The gages are of the flush front type, 10-in. in diameter and with chromium-plated brass cases. They consist of a recording water-pressure gage connected with the discharge pipe from No. 1 pump beyond the gate valve; an indicating water-pressure gage connected by a header to the discharge nozzles of the three units; an indicating steam-pressure gage connected with the steam header in the pump room; an indicating steam bleeder pressure gage connected with the header beneath the pump room floor; an indicating water vacuum gage connected by a header with the suction nozzles of the first-stage pump of the three units; an indicating steam vacuum gage connected by a header with the exhaust flange of each turbine; and an indicating mercury absolute pressure gage connected to the turbine exhaust header leading to the vacuum gage.

Regenerative Cycle. In order to obtain more useful work from the steam than would be secured in its normal passage through the turbine and condenser a 2-in. extraction steam line was tapped into the third stage of the turbines and carried to the open de-aërating, boiler feed water heater in the boiler room. The utilization of the heat in this steam after it had performed its major work in the turbines resulted in an economy approaching 5 per cent., depending on the amount of steam extracted, which in turn was governed largely by the pumping and generating load on the turbine.

Pumping Station Building. The pumping station and boiler room were built in 1876 at the very beginning of the water works. The buildings are of brick with slate roofs supported on wooden trusses. In 1904, after 28 years, the entire roof of the pumping station was raised 12 ft. to accommodate the vertical Barr steam pumping engine installed at that time. Last year, after a further lapse of 29 years, the roof was lowered the same 12 ft. because the added height was undesirable for steam turbine units.

The alterations in the pumping station, boiler room and coal pocket, both inside and outside, were made under the supervision of Clifford S. J. White, Architect, and by Richard White Sons, Inc., Contractor, both of Newton. Extensive changes were made in the interior both as to structural design and architectural finish. The pump room is well adapted to its function and presents an attractive appearance. The floor is concrete covered with red tile laid in cement mortar. The lower portion of the walls is of light buff, glazed tile blocks and the upper portion of acoustic plaster.

Duty Tests. Witness tests were made of each pump at the De Laval shops before shipment in order to be assured that the guaranteed duties were likely to be obtained. After all three units were completely installed and had been in regular operation for several weeks, final acceptance tests were made on November 21, 23 and 25, 1933, with very satisfactory results. The duties in ft.-lb. per 1 000 lb. of steam with generator loads of about 115 k.w. were as follows:

	Foot pounds
Unit No. 1	188 013 000
Unit No. 2	194 060 000
Unit No. 3	191 220 000
Average of three units	191 100 000
Guaranteed average duty	179 400 000

Average test duty exceeds guarantee by 6.5 per cent.

Pumping and Generating Costs. Although records of normal operation are available only for a short period the cost of coal for pumping and generating is about as follows. The price of coal suitable for use with the pulverizers is now $4.18 per net ton. Cost of coal for pumping water equals $3.75 per m.g. against a dynamic head of 274 ft. or $1.37 per m.g. per 100 ft. Cost of coal for generating electric power is 0.415 cents per k.w.hr. at the switchboard.

The former Barr pump had a station duty of 114 000 000 ft.-lb. per 1 000 lb. of steam and the cost of pumping at the same price for coal would be $6.28 per m.g. The saving in coal with the new pumping equipment, based on the present water consumption of 5 m.g.d., amounts to $4 600 per year. At the same time the cost of coal for generating power and light is $4 200 per year as against a former cost for purchased electricity, after an allowance for operating a new flooding pump, of $17 200 per year. The saving in electricity is $13 000 per year and the total saving $17 600 per year. This economy will increase with the water consumption.

Construction Cost. The cost of the complete remodelling of the pumping station, pumping equipment, boiler plant, coal pocket, outside piping connections and appurtenances including architect's fee, engineering and miscellaneous expenses was about $200 000.

BOILER PLANT
NEEDHAM STREET PUMPING STATION.

BY DAVID EAMES.*

[*Read January 10, 1934*].

The boiler plant of the Newton pumping station was completely modernized with new equipment without interrupting operation and without making any change in the building except to lower the floor level by about one foot.

The roof of the building is supported on two timber trusses dividing the width of the building into three equal bays. The headroom under the trusses of 18½ ft. was not sufficient for the boilers, so the arrangement adopted was to locate one boiler in each outside bay and the auxiliary equipment in the central bay.

The boilers are supported on steel columns independent of the building. These columns were designed heavy enough to carry also the overhead coal bunkers and the elevated feed-water heater platform as well as the access galleries at the sides and top of the boiler. The entire floor space was thus left clear for equipment.

The original square brick chimney, 105 ft. high, was not changed and is being used with complete success for the new boilers. The limited draft from this low chimney required careful consideration, however, in working out the design.

The capacity of the boiler equipment was determined by reference to the steam demand of the turbine when driving the largest of the three pumps and with full electrical load on the generator. Each of the two new boilers has capacity for this load so that only one boiler is in operation at any time, the second being in reserve. The full steam demand on the boiler including the main turbine and all auxiliaries is between 9 000 and 10 000 lb. per hour.

The boilers are watertube, of the cross drum straight tube type with 2 500 sq. ft. of heating surface, and are set 10½ ft. high from the floor to the bottom of the front header. The boilers have superheaters designed to give about 150° F. superheat to the steam. The operating steam pressure is 240 lb. per sq. in. and the total temperature of the steam is 550° F. which is 200° F. higher than the steam temperature of the original plant.

No attempt was made to secure high overload capacity from the boilers. For one reason, the available draft from the chimney is insufficient for high ratings, and for another reason, the small boilers and furnaces such as we have here, are best suited to conservative operating rates. When supplying the maximum steam demand of 10 000 lb. per hour the boiler will be operating at 135 per cent. of rated capacity.

*Consulting Mechanical Engineer, Boston, Mass.

The boiler feed-water is drawn direct from the river through the intake which was used for the same purpose by the original plant. The water is handled by small motor-driven pumps, in duplicate, and after leaving the pumps, passes through the steam-jet air-pump condensers at the main turbines where it reclaims several degrees of heat which would otherwise be lost. The water next passes through a closed-type feed-water heater which uses exhaust steam from the boiler feed pumps. The feed water then goes to the open de-aërating heater and thence by gravity to the boiler feed pumps. The open heater is supplied with steam extracted from the main turbines which has consequently done a certain amount of useful

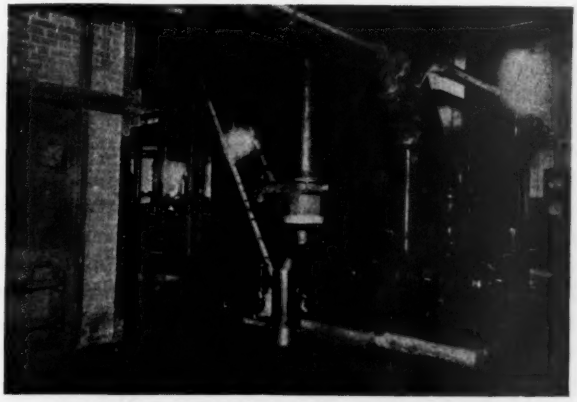

FIG. 1.—BOILER ROOM OF PUMPING STATION.

work before entering the feed-water heater. The pressure of the extraction line varies from atmospheric up to 5 or 6 lb. per sq. in. depending on which pump is running and what the load is. The feed-water temperature entering the boiler ranges from 212 to 225° F.

The steam leads from the boilers to the turbine room are 4 in. in size, and the throttle connection for each machine is 3 in. in size. The two 4 in. leads form a loop through the turbine room, with all stop valves conveniently accessible from the top of switchboard enclosure.

To go back to the fuel supply for the station, I think we would have decided to use oil had it been available at a price comparable with coal. As this was not the case, coal was chosen. The station has a large covered coal pocket dug into the side of the hill so that the coal is unloaded from an overhead gallery. This is the same arrangement as has always been in effect except that a platform scale has been installed on the gallery for weighing all coal as received.

The mechanical coal-handling system starts with a hopper at the boiler-room side of the coal pocket. Coal is wheeled to this point. After

passing through a grizzly to exclude lumps, it is then raised to upper part of the coal pocket by a bucket elevator which discharges to a screw conveyor. This in turn passes horizontally into the upper part of boiler room where it discharges into a small hopper located above the scale in which the coal is automatically weighed and from which it is discharged into the bunkers below. To prevent over-filling, the hopper over the scale is fitted with an automatic arrangement so that when it is nearly full the starter on the driving motor is tripped out and the entire conveying system shut down. This automatic feature is made use of three times a day by the different shift firemen in their regular filling of the bunkers.

Coal, after leaving the scale, is diverted by a flap damper into either of the two bunkers, one for each boiler. The shape of these bunkers was determined after careful study so that there would be a minimum of trouble due to arching of coal. The fuel has great tendency to do this, especially when wet. The bunkers, as built, have two adjacent sides vertical from top to bottom. The other two sides slope at 60 deg. from the horizontal. The corner between the vertical sides is rounded to a radius of 6 in., exactly conforming to the shape of the gate at the bottom. The results with these bunkers have fully met the expectations of their design.

A controlling feature in the design of the boiler room was to produce something which would be in keeping with the high-grade pumping equipment which was under consideration. This purpose found its fullest expression in the selection of pulverized coal as the method of firing. This is the cleanest, most efficient, and most up-to-date method of burning coal, giving boiler efficiencies of 78 to 80 per cent. against 71 to 72 per cent. for a stoker of type suitable for this plant. This represents at Newton a yearly fuel saving of $1 400 whereas the extra cost of the pulverizing equipment was $700, both values being stated in round numbers. The operation at Newton shows continually the three points which indicate high boiler efficiency, *viz.* high CO_2, low exit-gas temperature and a clean stack. A very important feature of a pulverized-coal installation is the design of the furnace. At Newton the furnaces are of the solid-wall type, without air or water cooling, and are amply large for the expected capacities. After about 6 months of service they show no spalling or other signs of failure. The new equipment is being operated by exactly the same men who formerly shovelled coal in the old plant, and there has been comparatively little to criticise in the way they have taken hold of the new ideas. I feel safe in saying that on the whole, the pulverized-coal system is proving eminently satisfactory, in the Needham Street Pumping Station.

EXPERIMENTS ON THE NEWTON WATER AND TREATMENTS FOR THE CONTROL OF CORROSION.

BY F. WELLINGTON GILCREAS.*

[*Read January 10, 1934.*]

The water supply of the city of Newton, Mass., is obtained from ground-water sources. The collecting system, consists of two large dug wells and a wooden conduit laid in the ground water, and with a number of driven wells discharging into it.

Effect of Infiltration Basins on Ground-water Quality. A preliminary study of the sources and quality of this supply indicated that certain of the sources were at times of unsatisfactory quality. These were abandoned, the resulting reduction in the quantity of water making it necessary to develop other sources of supply or methods of increasing the yield of the existing sources. Plans were made to raise the ground-water yield by the construction of infiltration or charging basins which would be filled with water from the nearby Charles River. Other papers in this Journal describe the construction and operation of these basins, as well as the increased yields of the various sources of supply resulting from the infiltration process. Because of the poor quality of the Charles River water, it was considered advisable to study the effect of the effluent from the infiltration basins on the quality of the ground water before developing the infiltration plan completely. For this purpose a number of test wells were located at various points around the basins. These were 2½-in. driven wells 50 ft. in depth. Samples were drawn from these test wells by means of an ordinary pitcher pump, the well being pumped to waste for some time before samples were collected to assure a representative sample of the ground water.

Studies of these test wells and also of the dug wells from which the supply is obtained, were continued for several years after the basins were constructed. When the basins were first filled, samples were collected from all sampling stations at frequent intervals, the sampling periods being extended as time went on. The results of these many analyses indicate that the natural soil was most efficient as a filtering medium, removing the color, bacteria and other unsatisfactory constituents of the river water very satisfactorily.

The results of the analyses of samples from the test well nearest to Basin No. 1, namely test well No. 8–28, may be considered representative of the results obtained at all the sampling points during this study. Figure 1 indicates the changes in the critical constituents of the ground water during the months of the years 1928–1929 when Basin No. 1 was in use. During

*Chemist, Weston and Sampson, Consulting Engineers, Boston, Mass.

the winter months, the basin was not in use and no samples from the test wells were collected; hence the gap in the curves. Samples were collected from this well before the basin was filled, the broken line indicating the amounts of the various constituents present in the ground water before infiltration started. The interesting features of this study are the rapid increase in dissolved oxygen and the corresponding decrease in the carbon dioxide content of the ground water shortly after the basin was filled. In

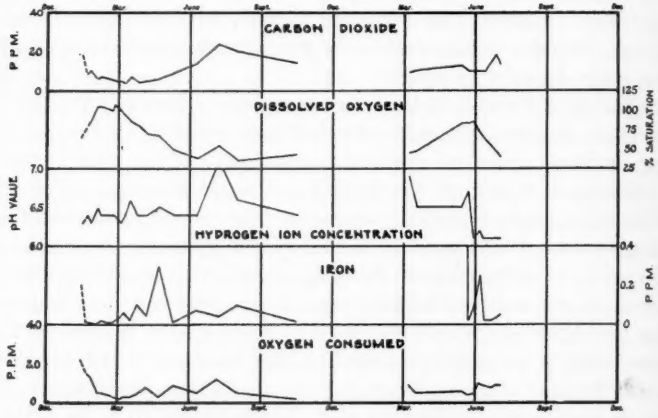

FIG. 1—CHARACTER OF WATER DRAWN FROM TEST WELL NO. 8-28 IN INFILTRATION STUDIES AT NEWTON.

fact, the amounts of these two gases in the ground water were very nearly the same as in the river water, presumably indicating a rapid passage of water from the basin through the gravel soil to the test well. Yet the amount of organic matter in the ground water, as shown by the oxygen consumed values, remained low. During the entire period of the study, the color of the ground water was zero, while that of the river water averaged 60 p.p.m. Bacteriologically, the water from the test well was nearly sterile, the total numbers of bacteria being low and gas-forming bacteria always absent, although the water pumped to the basins contained relatively large numbers of bacteria with gas-forming bacteria and *Bact. coli* present in practically all 10-c.c. portions tested. As the sides and bottom of the basin sealed up with silt, etc. from the river water, with a corresponding decrease in the rate of filtration, the dissolved oxygen and carbon dioxide values gradually changed to values nearer those of the ground water originally. However, the flow of the well continued to be abundant. At times the elevation of the water rose to a point 4 ft. above the surface of the ground. The amount of organic matter continued very low and bacteriologically the water was of excellent quality during the entire period studied. Conditions

in this well were representative of the other test wells except that the influence of infiltration was felt less and after longer periods of time in the wells at greater distance from the basin.

To determine, if possible, how far from the basin, water unsatisfactorily filtered could be found, another series of test wells was driven. The well previously referred to (No. 8–28) was about 150 ft. from the nearest edge of Basin No. 1, and of course at a lower elevation. On a direct line to Basin No. 1, test well No. 8–28B was driven at a point 100 ft., and No. 8–28A at a point 50 ft. from the basin, both being at about the same elevation as test well No. 8–28. These were also 2½-in. wells and driven to a depth of 50 ft. A study of the results, given in Table 1, shows that the gravel soil was filtering the river water satisfactorily; even at a distance of only 50 ft. from the wall of the infiltration basin. Practically all of the objectionable constituents were removed, the color of the water in this well being zero. Bacteria were not completely removed at a distance of 50 ft., although gas-forming bacteria were continuously absent in this well; but in 100 ft. bacterial removal was satisfactory.

At points nearer the basin, the dissolved oxygen became more nearly the same as that of the river water, absorption by the soil being less complete. From these studies it seemed reasonable to assume that infiltration basins located 100 ft. or more from a source of ground-water supply, and with a coarse gravel soil of the type in the Newton Water Department Reservation, would increase the yield of ground water without any deleterious effect on the quality of the ground water.

As additional infiltration basins were built to increase the yield of the wells and conduit that supply the city, the study of the effect upon the ground water and the general supply of such infiltration was continued. The results were about the same in all cases as those shown in test well No. 8–28. The yield of supply well No. 2 was nearly doubled by means of infiltration, but the only change in the character of the water was a lowering of the carbon dioxide value and a corresponding increase in the dissolved oxygen value. When the basins were not in use, the dissolved oxygen and carbon dioxide gradually returned to their original values. With the basins built along the line of the wooden conduit serving to increase the ground water yield in that location, very little effect upon the character of the water found in the conduit could be detected, although close check of the quality of the conduit water was maintained for several years. The yield increased greatly, and the only change detectable was a slight but decided increase in the temperature of the water. The dissolved oxygen and carbon dioxide content of the water changed but little during periods of infiltration.

Corrosion Studies. The process of infiltration had provided a sufficient supply of water of low color and excellent appearance and of a satisfactory bacteriological quality. However, the amounts of dissolved oxygen and carbon dioxide had not been changed appreciably and were still such as to cause serious action of the water upon service pipes. Complaints of dirty

water, stained plumbing fixtures, and short life of service pipes, particularly brass pipes, continued to be received. To determine more exactly the effect of the Newton water supply upon various types of service pipes, and

TABLE 1.— INFILTRATION STUDIES, NEWTON WATER DEPARTMENT.
Unless otherwise stated, results of analyses are expressed in parts per million.

	Test Well No. 8-28	Test Well No. 8-28B	Test Well No. 8-28A	Charles River
Distance from Basin—ft.	150.	100.	50.	....
Turbidity	0.	0.	1.5	3.
Color	0.	5.	0.	70.
Chlorides	5.5	6.5	5.5	6.0
Oxygen Consumed	0.35	0.85	0.4	6.8
Alkalinity	14.4	16.0	16.0	14.0
pH Value	6.4	6.4	6.3	6.7
Iron	0.04	1.6	0.03	0.5
Carbon Dioxide	5.2	7.0	6.0	6.0
Dissolved Oxygen — per cent saturation	95.0	66.4	89.7	94.4
Bacteria — 37°C. — 24 hr.	3.	7.	196.	165.
Bacteria — 20°C. — 48 hr.	3.	7.	500.	138.
Gas Forming Bacteria in 5 10-cc. samples	negative	negative	negative	4 positive

also to accumulate data upon which to base possible methods of treatment of the supply, a series of experiments was planned and carried on for a period of about one year.

In these experiments, the pipes used were wrought iron, galvanized iron and brass (60–40) pipes and copper tubing, 50-ft. lengths of regular service pipe size being used in each experiment. Four separate experiments with each type of pipe were planned, as follows:

(1) Untreated water.
(2) Regular water supply treated with soda ash to give a pH value of about 8.0.
(3) Aërated water.
(4) Aërated water treated with soda ash to give a pH value of about 8.0.

In order to simulate as nearly as possible the conditions actually obtaining in the average house, it was decided to pass through each experimental pipe 200 gal. of water per day, that being about the average volume of water used per domestic taker in Newton. For ordinary domestic use, the water is run at a rapid rate for a few minutes, then allowed to stand for varying periods in the pipe. As it was not possible to duplicate these conditions exactly, the experiments were planned to run for 12 hours each day, the 200 gal. flowing at an even rate through each section of pipe during that period. For the remaining 12 hours of the day, the flow was shut off

and the pipes allowed to stand full. This permitted the collection of samples representative of conditions of standing and ordinary use. Sampling taps were installed at suitable points in the pipe, and the pipes were slightly elevated to permit them to become filled and remain free of air pockets during the periods of standing. At first meters were installed in each line to measure the amount of water passing through each pipe, with ordinary valves to regulate the flow. Because of the slow rate of flow the meters would not register accurately, tended to impede the flow and were later removed. Also the valves were very unsatisfactory for regulating

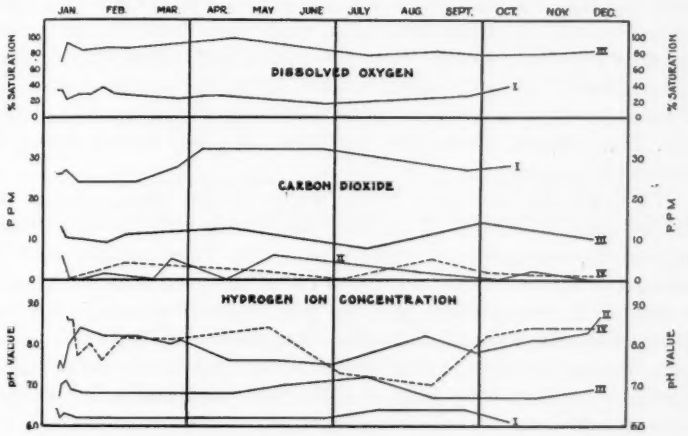

FIG. 2—CHARACTER OF WATER IN EXPERIMENTS ON THE CORROSION OF SERVICE PIPES.

Experiment I.—Untreated water.
Experiment II.—Raw water treated with soda ash (pH 8.2).
Experiment III.—Aërated water.
Experiment IV.—Aërated water treated with soda ash (pH 8.2).

such low flows. For the results of these experiments to be comparable, it was necessary that there should be a uniform flow of water through each pipe, and that the operation of the experimental plant should be constant. In order to maintain a uniform rate of flow through the pipes, Mr. W. S. Burke of the pumping-station staff designed and made orifice disks that could be fitted into the discharge ends of the pipes. These of course had to be different as the size of the orifice depended upon the pressure of the water, which varied considerably among the four experiments. The soda ash was fed into the water by gravity, using constant head tanks and similar orifice disks to keep the flow constant. These ingenious devices kept the flows constant and thereby permitted regular and constant treatment of the water. Aëration was accomplished by spraying through a per-

forated cap into a tank, from which the aërated water flowed by gravity to the proper grid of pipes.

Mr. Burke's faithful attention to the operation of the plant and his care in collecting the samples did much to insure the accuracy of the results.

The controlling factors in the action of water upon metal are the dissolved oxygen, the amount of carbon dioxide present and the pH value which is a function of the concentration of carbon dioxide. In these experiments, then, it was necessary to control these factors or to know their value at all times. Fig. 2 shows the variation in these substances during the year covered by the experiments. The supply being a ground water with a nearly constant temperature, seasonal variations in oxygen and carbon dioxide content are slight. Infiltration made very little change in these values, as previously stated. Thus the oxygen and carbon dioxide values in Experiment I are practically constant during the experimental period. Insofar as possible the pH value of the treated water was kept at a constant point. This diagram shows the variations in these factors during the experimental period.

To determine the effect of the water on the pipes, samples were collected under conditions of ordinary use, that is with the water flowing continuously, and under conditions of standing. These samples were then examined to determine the amount of metal dissolved from the pipes. In the case of the wrought-iron pipe, the iron naturally present in the water was determined and a correction made. Samples were collected at intervals of one, four and seven days, then weekly for the first month and at monthly intervals after that. The results of the four experiments are shown in Figures 3 to 6.

Experiment I: Untreated Water as Supplied to the City. Under conditions of use, there was a decided action on the wrought iron pipe at first, but this dropped off after a few weeks, so that but little metal was removed by the flowing water. The action was much more pronounced with the standing samples, but this also decreased as a protective coating of ferric oxide was built up on the walls of the pipe. These results are in accord with experience with the action of corrosive waters on iron pipes. Probably the condition here is similar to that existing in the distribution mains in the city. Occasionally in the experimental unit portions of the deposited ferric oxide would slough off, a condition that also occurs at times in the distribution system, accounting in part for the trouble encountered with dirty water.

With the galvanized iron pipe, action of the flowing water was slight, but under conditions of standing, the solution of large amounts of zinc continued throughout the experiment, although with some irregularities that are hard to explain. With the brass pipe, solution of the zinc under conditions of standing continued with little change throughout the experiment. The flowing water had but little effect. The interesting feature in the experiment is the increased removal of zinc after several months' use.

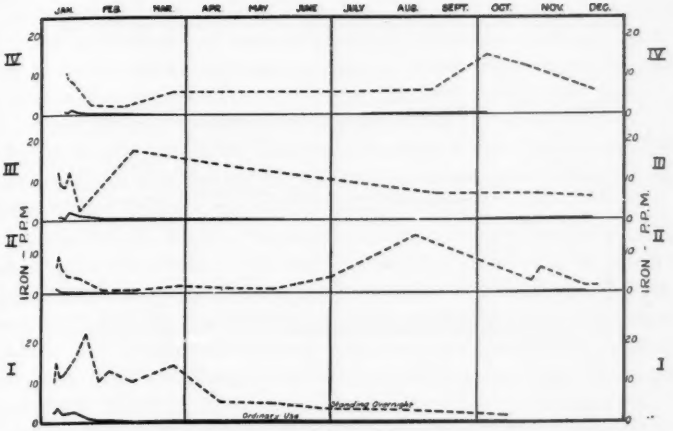

FIG. 3—CORROSION OF WROUGHT IRON PIPE INDICATED BY SOLUTION OF IRON.

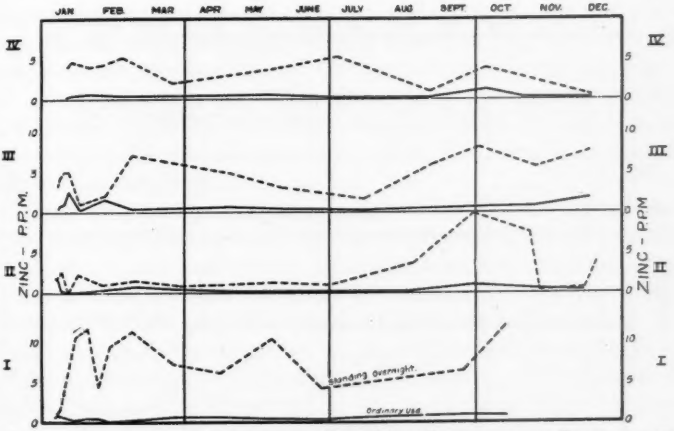

FIG. 4—CORROSION OF GALVANIZED IRON PIPE INDICATED BY SOLUTION OF ZINC.

Experiment I.—Untreated water.
Experiment II.—Raw water treated with soda ash (pH 8.2).
Experiment III.—Aërated water.
Experiment IV.—Aërated water treated with soda ash (pH 8.2).

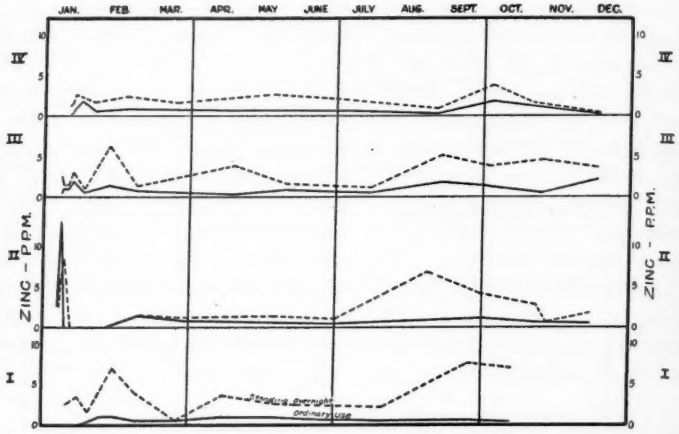

FIG. 5—CORROSION OF BRASS PIPE INDICATED BY SOLUTION OF ZINC.

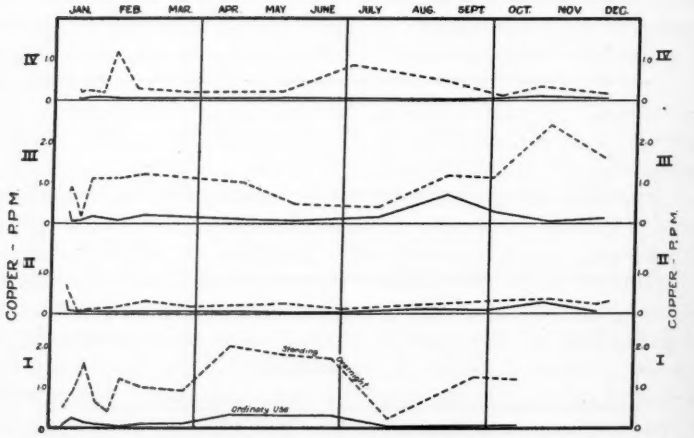

FIG. 6—CORROSION OF COPPER TUBING INDICATED BY SOLUTION OF COPPER.

Experiment I.—Untreated water.
Experiment II.—Raw water treated with soda ash (pH 8.2).
Experiment III.—Aërated water.
Experiment IV.—Aërated water treated with soda ash (pH 8.2).

These results seem to be quite in accord with the general experience with brass pipe in the city and offer some explanation of the short life of brass in most houses in Newton. With the copper tubing, there was also extensive and continued solution of the metal under standing conditions, with appreciable action under flowing conditions. These results were also quite in accord with general experience in the city, and explain the green stains on plumbing fixtures where copper pipe was used.

As previously explained, Experiments II, III and IV were conducted to determine the most satisfactory treatment to prevent the action noted in Experiment I on the various types of pipe metals.

Experiment II: Water Treated with Soda Ash. In Experiment II, the regular supply was treated with soda ash to give a pH value of about 8.0. Soda ash forms no protective coating in the pipe but by increasing the pH value acts to decrease the velocity of the chemical action of the oxygen in the water on the pipe and by reducing the concentration of the carbon dioxide reduces the solvent action of that substance upon zinc and copper. The results of this experiment show just that action. Corrosion of the iron pipe was reduced under conditions both of standing and use with the exception of a high value in August, when due possibly to temperature changes deposited iron sloughed off, as often happens in practice. Except for a brief period toward the end of the experiment the galvanized pipe was protected by treatment. With the brass pipe, the treated water dissolved substantial quantities of zinc at the start of the experiment. But this solvent action decreased very quickly, and action on the metal was slight under conditions of standing and flow, except for a brief period at the latter end of the experiment. There seems to be no explanation for this sudden increase in the amount of zinc dissolved. With the copper tubing, treatment provided especially good protection, only very small quantities of copper being dissolved.

Experiment III: Aërated Water. In Experiment III, the carbon dioxide and the resulting hydrogen ion concentration were reduced by aëration which, however, increased greatly the concentration of dissolved oxygen. As would be expected with higher concentration of oxygen, the action of the water in the iron pipe was much more pronounced especially under standing conditions. The action on the metal decreased more slowly with time than in Experiment I, indicating less tendency for the deposition of a protective coating of ferric hydroxide. The lower value of carbon dioxide reduced slightly the action of the water on the galvanized pipe, less zinc being dissolved than in Experiment I.

The lower concentration of carbon dioxide resulting from aëration reduced slightly the solvent action of the water for the zinc and copper of the brass and copper pipes. However, a concentration of 10 p.p.m. of carbon dioxide with the increased concentration of oxygen seemed to be sufficient to cause nearly as much action as with the un-aërated water with 25 p.p.m. of carbon dioxide. Results obtained in other investigations with

Newton water would indicate that with concentrations of carbon dioxide exceeding 6 to 7 p.p.m. action on copper with resulting stains on plumbing fixtures might be expected.

Experiment IV: Aërated Water Treated with Soda Ash. In Experiment IV, aërated water was treated with soda ash to give the same pH value as in Experiment II, namely about 8.0. As would be anticipated the treatment reduced the action of the water on the iron pipe slightly over that in Experiment III. The reduction in the concentration of carbon dioxide accompanying the increased pH value decreased the action of the water on the other pipes. However, perhaps because of the increased oxygen concentration, protection was not quite equal to that in Experiment II.

These experiments demonstrated that the Newton water supply as received at the station has a decided effect upon the metals constituting the ordinary types of service pipe. Protection of these pipes could be secured by increasing the pH value of the water by the use of soda ash. Reduction of the carbon dioxide content by aëration gave no protection; in fact the action on the iron pipe was increased by the greater concentration of oxygen. Aëration followed by an increase of the pH value failed to give as good protection as similar treatment of the un-aërated water, where the concentration of dissolved oxygen was low.

Because of the relatively short time covered by the experiments examination of sections of the pipes removed after the conclusion of the experiments gave but little information in addition to that obtained by the analyses of the samples of water. The iron pipes showed the presence of a deposit of ferric oxide. The galvanized pipe, especially in Experiment I, showed the result of the solvent action of the water upon the zinc. The copper and brass pipes gave little visual sign of corrosion.

Use of Lime. As stated before, soda ash produces no protective coating as a result of its reaction with the carbon dioxide present in the water. Lime, on the other hand, does react with carbon dioxide to precipitate a coating of calcium carbonate on the walls of the pipe which acts as a barrier to prevent further action of the water on the metal. To insure the insolubility of the calcium carbonate, however, pH values higher than 8.0 must be maintained. Addition of lime causes substantial increase in the hardness or soap-consuming power of the water. While the hardness normally present in the water supply at Newton (about 35 p.p.m.) is not high, it is enough greater than that of Boston Metropolitan water to be the cause of some complaint by citizens. In order to avoid increase in the hardness the experiments were conducted using soda ash instead of lime. While the cost of lime treatment would be much less than that of soda ash, the additional cost of soap used by the people would be much larger than the saving in the cost of chemicals. The matter of treating the aërated water with lime was also studied, but it was found that the cost of aëration would be greater than the saving in the cost of chemicals.

Furthermore, the increased concentration of oxygen that would result

from aëration was undesirable, in view of its action on the iron pipe in the experimental work. The higher pH value that would be necessary, if lime was used, in order to insure insolubility of the calcium carbonate coating was about what would cause precipitation of manganese, especially with the higher concentrations of oxygen following aëration. Since the Newton water contains appreciable quantities of manganese, the possibility of trouble with precipitated manganese was to be avoided.

Treatment Adopted. In view of these facts and on the basis of the results of the experimental work, it was decided to treat the supply with soda ash to reduce the aggressive action of the water toward metals. It was planned to so treat the water as to maintain the pH value at a point no higher than 8.0, and no lower than 7.6. The upper limit would not cause the precipitation of dissolved manganese, while pH values less than 7.6 would not adequately protect brass and copper. It was also feared that pH values higher than 8.4 might disturb deposits already existing in the mains, with a resulting period of dirty water.

In December 1931, treatment of the supply was started. In order to run no risk of disturbing deposits in the supply mains by too sudden change in the reaction of the water, the pH value was increased very gradually. At first only enough soda ash was introduced to increase the pH value from 6.2, the normal value for the water, to 6.4. This value was maintained for about four weeks. Sampling stations were established in various parts of the city on flowing mains, on dead ends and at the extreme ends of the system to check by analyses the effect of the treatment on the quality of the

TABLE 2.—EFFECT OF TREATMENT WITH SODA ASH, NEWTON WATER DEPARTMENT. Unless otherwise stated, results of analyses are given in parts per million.

	Pumping Station.		Supply Yard.
	Untreated	Treated.	
Turbidity	0.	0.	0.
Color	2.	5.	5.
Alkalinity	26.0	74.0	72.0
Hardness	29.9	29.9	31.2
pH Value	6.2	7.8	7.6
Iron	0.08	0.10	0.08
Carbon Dioxide	25.0	4.0	4.1
Dissolved Oxygen — Per cent saturation	33.8	....	....

water. The pH value was then increased slowly by increments of 0.2 pH to the maximum of 7.8 over a period of six months. Samples were collected from each of the sampling stations just before each increase in the dosage of soda ash. In this way a complete check was kept on the effect of the treatment. No trouble was encountered and no complaints of dirty water were received during the period of establishing the treatment, indicating

that the increase in the alkalinity of the water did not disturb deposits already existing in the mains. When the pH value increased to a point above 7.2 complaints of green stains due to the action of the water on copper ceased. In several instances the Water Department received comments to the effect that these stains had ceased and asking why. Treatment of the supply at the pH value decided upon has continued since May 1932, with uniformly good results. Complaints of dirty water, except on dead ends, have decreased markedly. Complaints of green stains due to corrosion of copper have entirely ceased. Of course treatment has not been carried on long enough for all complaints of the collapse of brass pipe to cease. However, it is anticipated that the life of brass service pipe in Newton will be much extended from now on.

Table 2 gives typical analyses of the water at the pumping station before treatment, after treatment with soda ash, and from the tap at the Water Department Supply Yard in Newtonville, the latter representing the character of the water as taken from the distribution system. As treated at present, the carbon dioxide is reduced from 25–30 p.p.m. to 4.0 p.p.m.; the pH value is increased from 6.2 to 7.8. To do this requires the addition of approximately 400 lb. of soda ash per million gallons. With soda ash costing 1.25 cents per pound, this is equivalent to a cost of $5.00 per million gallons of water pumped. During the course of a year, this amounts to about $10 000. However, the improved character of the water and the reduction in its corrosive character compensate for the expenditure.

PROCEEDINGS.

DECEMBER MEETING.

STATLER HOTEL, BOSTON, MASS.

Wednesday, December 13, 1933.

Vice-President Howard M. King in the Chair.

THE CHAIRMAN. Gentlemen, at the last meeting of this ASSOCIATION Sir Alexander Houston, a noted English engineer, was made an honorary member of this ASSOCIATION. He has since passed away, and I ask you to rise as a mark of respect.

Secretary GIFFORD. At the Executive Committee meeting this morning Mr. J. Arthur Jackson of Portland, Maine, was elected a member, and Mr. M. N. Baker of New York was reinstated.

I have a report from the Committee on Revision of the Summary of Statistics, which I will read:

BOSTON, December 13, 1933

Executive Committee, New England Water Works Association.

Gentlemen: Your committee on revision of present form of "Summary of Statistics" as adopted by the ASSOCIATION in 1902, presents the accompanying report showing the proposed revised form.

We desire to acknowledge the help that we have received from the members in compiling this form and recommend that it be set up and copies made available for those who may desire them.

We also recommend that final vote on adoption by the Association be postponed until the next regular meeting, after due notice has been given in the regular calendar.

Respectfully submitted,

(Signed) STEPHEN H. TAYLOR
HERVEY A. HANSCOM
SAMUEL E. KILLAM,
Chairman.

A paper "Rigid Service Connections to Main in Preference to Goosenecks," was read by Michael J. Herlihy, Foreman, Newton Water Department, Newton, Mass. Messrs. Roger W. Esty, Henry T. Gidley, David A. Heffernan, Nelson Boardman, Hervey A. Hanscom, Frank S. Brainard, Edmund C. Sanderson, and John S. Caldwell took part in the discussion.

A paper "Water Works Practice at Taunton, Mass.," was read by Arthur C. King, Superintendent and Clerk, Taunton Water Works, Taunton, Mass. Mr. Roger W. Esty discussed the paper.

A paper "Results of a Water Waste Survey in Lexington, Mass.," was read by Albert A. Ross, Superintendent, Water and Sewer Department, Lexington, Mass. The discussion was participated in by Messrs. Henry T. Gidley, Patrick Gear, Richard H. Ellis, and David A. Heffernan.

The following members took part in a discussion of the N.R.A. and C.W.A. programs as applied to Water Works: Frank A. Gifford, Harry F. Beal, Roger W. Esty, Edmund C. Sanderson, Albert A. Ross, Patrick Gear, Donald C. Calderwood and Henry T. Gidley.

[*Adjourned.*]

JANUARY MEETING.

HOTEL KENMORE, BOSTON, MASS.

Wednesday, January 10, 1934.

President E. Sherman Chase in the Chair.

Secretary GIFFORD. At the meeting of the Executive Committee this morning the following were elected members of the Association.

Lorenzo B. Fortin, Water Commissioner, Greenfield, Mass.; Alfred A. Lamarine, Superintendent Water Works, Wayland, Mass.

A paper "A Resumé of Changes in Practice and Recent Improvements in the Newton Water Department," was read by Richard H. Ellis, Water Commissioner, Newton, Mass.

A paper "Reconstruction of the Pumping Station and the Development of the Source of Supply at Newton, Mass.," was read by George A. Sampson, Consulting Engineer, Weston and Sampson, Consulting Engineers, Boston, Mass.

A paper "The Boiler Plant at the Needham Street Pumping Station," was read by David Eames, Consulting Mechanical Engineer, Boston, Mass.

A paper "Experiments on the Newton Water and Treatments for the Control of Corrosion," was read by Wellington Gilcreas, Chemist, Weston and Sampson, Consulting Engineers, Boston, Mass.

A short discussion of the papers was participated in by David A. Heffernan, Warren A. Gentner, and Robert Spurr Weston.

FEBRUARY MEETING

HOTEL KENMORE, BOSTON, MASS.

Wednesday, February 14, 1934.

President E. Sherman Chase in the Chair.

Secretary GIFFORD. At the Executive Committee meeting this morning William J. McCarthy, Foreman Water Department, Newton, Mass., was elected to membership.

The PRESIDENT.

I have been asked to call to the attention of the ASSOCIATION House Bill No. 693 which reads as follows:

Chapter ninety-one of the General Laws is hereby amended by adding at the end thereof the following new section:

Section 60. It shall be lawful to swim or boat, or except as otherwise provided in sections twenty-seven, thirty-seven, thirty-eight, thirty-nine, fifty-four, and sixty of chapter one hundred and thirty-one to fish in any lawful manner at any lawful time, in any of the fresh waters of this Commonwealth, other than artificial reservoirs; provided that in so doing no trespass is committed and no pollution dangerous to public health in fact results. Any rule, regulation or order of any public board or officer, or any city or town ordinance, contrary to this section, shall be null and void.

STEPHEN H. TAYLOR. I would like to present the following resolution:

WHEREAS the enactment of House Bill 693, now before the Legislature of Massachusetts, would greatly increase the difficulty of protecting many of the public water supplies of the State against dangerous pollution, and might even make it impossible to prevent acts causing serious hazard of pollution, but where it might be difficult or impossible to prove that "polution dangerous to the public health, in fact, resulted," therefore

BE IT RESOLVED that the New England Water Works Association, in regular meeting assembled, February 14, 1934, notify the Legislative Committee on Water Supply of its opposition to the passage of this or any similar legislation.

(The question was put and the resolution unanimously adopted by a rising vote.)

SAMUEL E. KILLAM presented the outline of the new form for water-works statistics for discussion.

A paper "The Boston Metropolitan Water Supply Extension," was read by Karl R. Kennison, Designing Engineer, Metropolitan District Water Supply Commission, Boston. The paper was illustrated by stereopticon slides and moving pictures.

Written discussions were submitted by Frank E. Winsor, Robert Spurr Weston, and Francis H. Kingsbury, the latter representing Arthur D. Weston, Chief Engineer, Massachusetts Department of Public Health.

(*Adjourned*)

ALBERT LIVINGSTONE SAWYER.

This Association mourns the passing of Albert Livingstone Sawyer, an active member for thirty-four years, and Treasurer from September 30, 1924, until his death.

Mr. Sawyer was born in Haverhill, Mass. on July 27, 1864, the son of Harris and Abby Knapp (Livingston) Sawyer, and died, quite suddenly, in the house in which he was born, on September 25, 1933, after several months of ill health.

Mr. Sawyer graduated from the Haverhill High School in 1882 and in 1892 when the city of Haverhill took over the water works he became an inspector and later the Registrar and Clerk of the Board, which positions he held the rest of his life.

Water works administration was made his life study and in 1916 he was awarded the Dexter Brackett Memorial Medal of this ASSOCIATION for the most meritorious paper presented during the year, the subject being "Some Advantages of a Classified Cash Book." He was an assistant assessor for several years and a member of the School Committee from 1904 to 1909, when the commission form of government was established.

Mr. Sawyer was an active member of the Trinity Episcopal Church, and for twelve years was Superintendent of the Sunday School. He also served as vestryman, treasurer and clerk, and for years was a delegate to the diocesan convention of the church. He had been a licensed lay reader since 1893, and as such had read the service in various churches of the diocese during the summer months, and was also treasurer of the Merrimack Branch of the Church School Union. He was greatly interested in the work of the Haverhill Boys Club and was Treasurer from 1909 until his resignation in 1932, accepting no compensation for this service. He was a member of the Episcopalian Club of Massachusetts, and the Haverhill Historical Society, of which he was Treasurer from 1924 to his death.

His loss will be keenly felt in every circle in which he had a part.

HERBERT C. CROWELL
JOHN CARROLL CHASE.

SIR ALEXANDER HOUSTON.

Sir Alexander Cruikshank Houston, Director of Water Examination, Metropolitan Water Board since 1905, died at Hampstead on October 29, 1933, at the age of 68. He was to have retired at the end of the year.

The son of Surgeon-General John Houston, he was educated at Merchiston Castle School, Edinburgh, and Edinburgh University, where he took his medical degrees in 1889. He was awarded the Gunning Victoria Jubilee Alison Prize in public health and medical jurisprudence in 1892 and graduated B.Sc., and D.Sc. He worked for the Local Government Board on lead poisoning due to moorland water supplies from 1893 to 1905, and on various other subjects connected with public health from 1897 to 1905.

For the London County Council he was engaged on sewage disposal experiments at Crossness and Barking in 1898–1900, and later on the bacteriology of watercress, milk and other foods. He was bacteriologist to the Royal Commission on Sewage Disposal, 1899–1905, and was one of the experts engaged in the arbitration proceedings arising out of the formation of the Metropolitan Water Board, and in the Belfast Health Enquiry, 1907. In 1907 he was appointed British representative on the International Commission of three experts to enquire into the quality of the Cairo water supply, and with the late Sir Alexander Binnie held a similar enquiry at Ottawa in 1913.

In addition to numerous reports Sir Alexander was the author of "Studies in Water Supply," 1914; "Rivers as Sources of Water Supply," 1917; and "Rural Water Supplies and their Purification," 1918. He had been an Honorary Member of the Institution of Water Engineers (England) since 1923 and of the New England Water Works Association since 1929.

His pioneer work in the chlorination of the Lincoln (England) water supply in 1904 led the way to the world-wide adoption of this method of disinfection for water supplies. His annual reports as Director of Water Examination of the Metropolitan Water Board, contain a vast amount of valuable routine and research data relating to the quality of water supply.

He was a man of broad culture as attested by the charm of the descriptive portions of his annual reports. To meet him was a delight and the friendliness and kindliness of his manner was such as to endear him to those with whom he came in contact. By his death the New England Water Works Association has lost one of its most distinguished members, and the world its foremost authority on water quality.

E. SHERMAN CHASE.

Most of the biographical material has been furnished by A. T. Hobbs, M. Inst. C. E. Sec'y. Inst. of Water Engineers, of England.

FLOYD A. NAGLER.

Floyd A. Nagler, member of this Association since 1919, died on November 10, 1933, following an emergency operation. Professor Nagler had been a member of the faculty of the University of Iowa since 1920, first as Assistant Professor of Mechanics and Hydraulics and then as Professor of Hydraulic Engineering. In 1931, due largely to his energetic work, the Iowa Institute of Hydraulic Research was created by the State Board of Education, and Professor Nagler became the first Director of the Institute.

His technical and scientific contributions number more than 50 significant articles dealing chiefly with his specialty—hydraulic engineering and hydrology. Three of his papers won for him prizes offered by the American Society of Civil Engineers.

In addition to membership in the New England Water Works Association he was a member of the American Society of Civil Engineers, Society for the Promotion of Engineering Education, American Association for the Advancement of Science, American Meteorological Society, American Water Works Association, Iowa Engineering Society and American Geophysical Union.

E. SHERMAN CHASE.

Data furnished by F. T. Mavis, Assoc. Director, Iowa Institute of Hydraulic Research.

New England Water Works Association

ORGANIZED 1882.

Vol. XLVIII. June, 1934. No. 2.

This Association, as a body, is not responsible for the statements or opinions of any of its members.

THE COBBLE MOUNTAIN POWER TUNNEL.

BY HARRY H. HATCH.*

[*Received January 29, 1934.*]

The Cobble Mountain development is unique in many ways. It is a combined water-supply and water-power project, costing about six million dollars, being built by a municipality. The watershed tributary to the project, is only 46 sq. miles yielding an average flow of about 90 c.f.s. Yet there is included the construction of the highest hydraulic fill dam in the world. The fall of head is 334 ft. minimum and 456 ft. maximum, with working average of about 420 ft. The power house and the power tunnel have a capacity of at least ten times the mean flow of the stream.

The city of Springfield has been able to sell the power at an annual figure of $270 000 for the next 30 years, without jeopardizing the full benefit of the ultimate development of the watershed for its municipal water supply purposes.

The Cobble Mountain project has been a happy combination both for the city of Springfield and the Turners Falls Power & Electric Company. The city would have been unable at this time to develop its watershed to full capacity without the financial returns through the power company; and the latter has acquired the most important water power within its territory to supply its badly needed requirements without any financial obligations other than the yearly payment of rental.

The Cobble Mountain tunnel (Fig. 1) is built especially for the development of water power. Its entrance is located 1200 ft. south of the spillway of Cobble Mountain Reservoir and follows the northern slope of Sweetman Mountain in an easterly direction to a point south of and almost directly above the power house. The tunnel will always be under pressure, and its course was laid to give at least 150 ft. cover on top, except near the end where a steel lining has been provided.

The present average daily water consumption of the city is about 28 c.f.s. The power company will draw this amount, or whatever the city requires for its daily use, every day during the peak-load hours. The city has more than one day's supply capacity in reservoirs or basins below the Power

*Engineer in charge Cobble Mountain Reservoir, Springfield Water Works, Springfield, Mass.

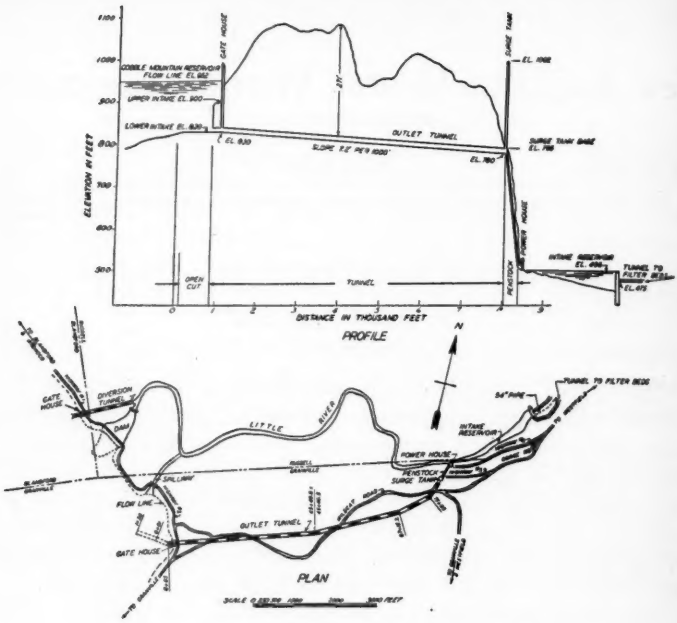

FIG. 1.—COBBLE MOUNTAIN POWER TUNNEL.
Location, plan, and profile.

FIG. 2.—POWER HOUSE DIRECTLY BELOW THE SURGE TANK.
Intake Reservoir of the Springfield Water Supply in foreground.

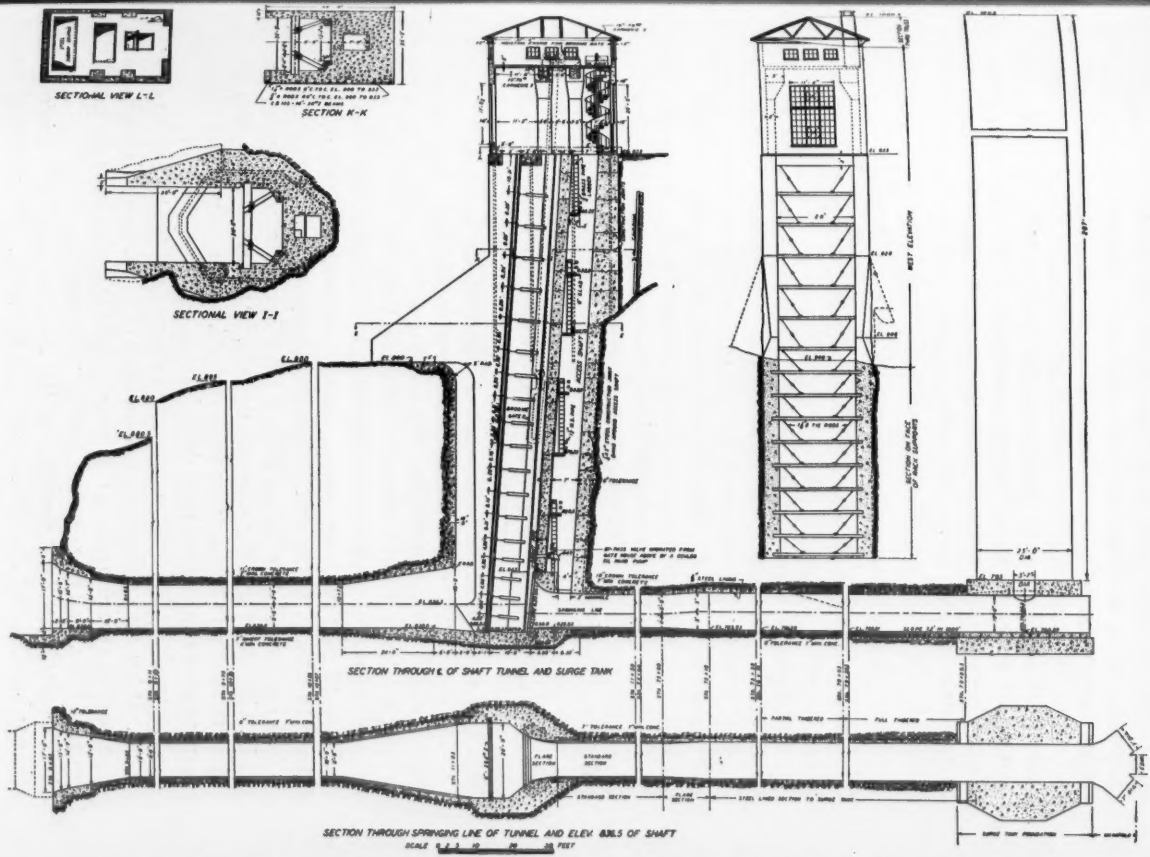
SECTIONAL VIEW L-L
SECTION K-K
SECTIONAL VIEW I-I
SECTION THROUGH ℄ OF SHAFT TUNNEL AND SURGE TANK
SECTION THROUGH SPRINGING LINE OF TUNNEL AND ELEV. AXIS OF SHAFT

FIG. 3.—COBBLE MOUNTAIN POWER TUNNEL. Typical sections of tunnel, shaft, gate house and surge tank.

House (Fig. 2), and the daily supply can be drawn any time during the 24 hours. The difference between the mean flow of the stream and the city's daily draft will be impounded in Cobble Mountain Reservoir for use, under certain restrictions, at the convenience of the power company. As the tunnel and the power plant capacities are ten times that of the mean flow of the river, the impounded yearly excess water can be drawn during a period as short as about five weeks. The power company will use this water during dry seasons when heretofore it had to operate steam plants to supplement its hydraulic plants.

Description of Tunnel, Shaft and Gate House.

The tunnel has two inlets with level grades, the lower at elevation 830 ft. and the upper at elevation 900 ft. (Fig. 3).

The approach channel of the lower inlet is through solid rock; length 450 ft., bottom width 15 ft., and side slopes 4 on 1. The lower inlet is circular in shape, 17 ft. in diameter, and is reduced to a 10 ft. by 11 ft.-6 in. section within a distance of 20 ft. This section is 168 ft. long and has 100 sq. ft. of area. It has a level grade with invert elevation at 830 ft. There follows a 30-ft. transition section, the end of which at the control shaft is 18 ft.-8 in. by 16 ft.-6 in. The portal is so arranged that it can be closed by an emergency hollow steel gate for the purpose of repairing the tunnel.

The upper inlet is 20 ft. wide at the bottom with a slope of 2 to 1 for dirt, and ¼ to 1 for rock excavation. Near the shaft there are two retaining walls as part of the concrete structure of the main shaft. These walls are 30 ft. long, 6 ft. high at the end, rising to a height of 28 ft. at the shaft.

The main shaft has an octagonal shape with about 390 sq. ft. of area from elevation 830 to 900 (Fig. 4). The easterly side, on which the gate will be operated, has a slope of 15.8 on 1. This side forms the 3-ft. thick partition wall between the main shaft and the 6 ft. by 7 ft. small shaft. The main shaft has 18 rack supports spaced, center to center, from 4.95 ft. at the bottom to 8.25 ft. at the top. These rack supports are made of 24-in., 70-lb. I beams with two 14-in., 58-lb. I beam horizontal knee braces, and 1⅛-in. diameter vertically diagonal tie rods. The trash racks which are bolted to these rack supports are made of ⅜-in. by 5-in. bars spaced 3⅜ in. center to center and extend from the bottom to the top of the shaft.

The small shaft is built to serve as an air vent and to give access to the tunnel. It has six landings and ladders made of ¾-in. diameter brass pipe rungs cast into the concrete. At its bottom, right at the crown of the tunnel, there is a 36-in. diameter manhole leading to the tunnel.

The gate house, built directly over the shaft, is a two story, reinforced concrete building, 28 ft. by 32 ft. in plan, and 45 ft. high. In it is installed the equipment for operating the Broome self-closing sluice gate, which is 12 ft. wide by 13 ft. high, weighing about 15 tons, and operated by a Philips & Davies 200-ton capacity electric-driven double drum hoist.

Near the manhole, in the access shaft there is a 12-in. bypass valve operated by a Goulds oil-pressure hand-pump in the gate house. With the Broome gate closed and the tunnel empty, this valve permits filling the tunnel with water from the power house to the shaft, thus equalizing the pressure on both sides of the gate so that it can be raised easily.

The area of the tunnel at the gate seat is 156 sq. ft. It is brought down within a distance of 6.7 ft. to a horse-shoe section which is 10 ft. by 9 ft.-4 in., has an area of 78.96 sq. ft. and is 6 376 ft. long. There follows a transition section of about 12 ft., changing the horse shoe into a 10-ft. diameter circle. In its terminal length of about 450 ft. the tunnel has a 7/16-in. thick

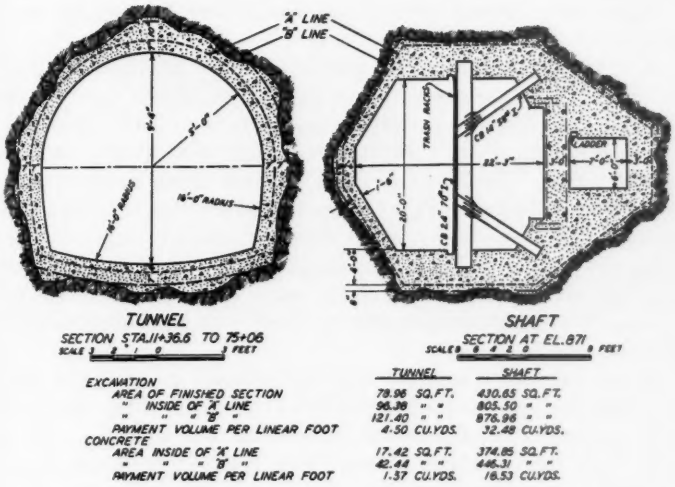
'A' LINE
'B' LINE
TRASH RACKS
LADDER
TUNNEL
SECTION STA.11+36.6 TO 75+06
SCALE 1 0 1 0 3 FEET
SHAFT
SECTION AT EL. 871
SCALE 8 4 2 0 8 FEET
TUNNEL SHAFT
EXCAVATION
AREA OF FINISHED SECTION 78.96 SQ.FT. 430.65 SQ.FT.
" INSIDE OF 'A' LINE 96.38 " " 805.50 " "
" " " 'B' " 121.40 " " 876.96 " "
PAYMENT VOLUME PER LINEAR FOOT 4.50 CU.YDS. 32.48 CU.YDS.
CONCRETE
AREA INSIDE OF 'A' LINE 17.42 SQ.FT. 374.85 SQ.FT.
" " " 'B' " 42.44 " " 446.31 " "
PAYMENT VOLUME PER LINEAR FOOT 1.37 CU.YDS. 16.53 CU.YDS.

FIG. 4.— TYPICAL CROSS-SECTIONS OF TUNNEL AND SHAFT.

and 10-ft. diameter steel lining, all joints welded, extending through the surge tank foundation. From the shaft to the surge tank there is a uniform slope of 0.72 per cent., and there occur three horizontal angles in the alignment. The total length of the tunnel is 7 100 ft.

The surge tank is 25 ft. in diameter and 207 ft. high. Its top is 50 ft. above the maximum flow-line of the reservoir. The tank is made up of riveted steel plates, varying from 1-in. thickness at the bottom to 7/16 in. at the top. The bottom of the surge tank is 5 ft. above the top of the steel lining of the tunnel. The two are connected vertically by a 5 ft.-7 in. diameter steel pipe. Near the surge tank foundation, the steel lining is connected into a manifold with three outlets, the middle one of 5-ft. diameter and the other two of 7-ft. diameter, leading into 3 penstock lines. The middle penstock is 5 ft.-6 in. in diameter with thickness of plate varying from 3/8 to 5/8 in. and the other two 7 ft.-9 in. in diameter with plates from

3/8 to 7/8 in. thick. The penstocks are about 660 ft. long with a 284 ft. drop in elevation. The power units consist of two 13 620 and one 5 760 k.v.a. generators.

Control of the Tunnel Lines and Grades.

The ends of the tunnel were located on the ground with the help of topographic maps, and careful surveys with tape and transit were made on the surface over the possible tunnel courses. The traverse was checked independently; the resulting error of closure was 1 in 15 000. Later on, the survey was also checked by a triangulation system, the difference being only 0.6 ft. in a distance of about 8 000. The few hundred dollars spent for the triangulation system was well worth the peace of mind that it produced.

Independent lines of levels, to the nearest 0.001 ft. were run from one portal to the other and back.

A triangulation station was established over the tunnel course near each heading, and permanent bench marks and targets were established far enough away from the portal so as not be to disturbed by blasting.

East Heading. About 4 500 ft. of the tunnel were driven from the east, or outlet, portal. The three horizontal angles in the tunnel were all within this distance. The line and grade were carried into the heading by precise taping for a distance of about 500 ft. from the nearby triangulation station, making use also of the permanent target and the bench mark set for this purpose. A hub was established on the tunnel center line a few feet in front of the portal. Its station was carefully determined and the necessary reference points put in.

As the excavation assumed a normal progress in the tunnel, the lines and grades were given to the heading boss at least once a day. Every 100 ft. a hole was driven into the crown of the tunnel and a wooden plug driven into it. Into the plugs were driven spads, from which plumb bobs were suspended for transit points. The spads were used also for turning points and bench marks. Ordinary transits and tapes were used for every day work. Line was carried ahead always by double centering. Before any angle in this drift was turned, the course was measured at least twice by precise taping. The precise work was done with just as little interference to the progress of construction as possible. For this reason the engineering party sometimes had as many as nine men. The equipment for precise work consisted of:

One Buff & Buff tunnel transit, with special attachments.
One 200-ft. steel standardized tape.
One 50-lb. spring balance.
Two 5-ft. long steel bars.
One Wider Thermometer, and extension tripod, level rod, plumb bob, flash lights, etc.

Corrections were made for absolute length, sag, temperature, and difference in elevation of the ends of the tape. The tape was standardized with a 35-lb. pull. The same pull was used in the taping so as to eliminate the correction for it.

The exterior angles were so small that the points of intersection remained within the tunnel section. At each angle point the angles were turned every day with moderate care until the heading advanced a few hundred feet to establish a good control in the new course. Then a set of 12 interior and 12 exterior angles was turned, and the mean taken, using just as long a back-sight as possible. This work was checked independently and the necessary adjustment was made in the next course.

West Heading. The contractor sank the shaft and started the easterly drift before boring through the section to the west, or inlet, portal. Both

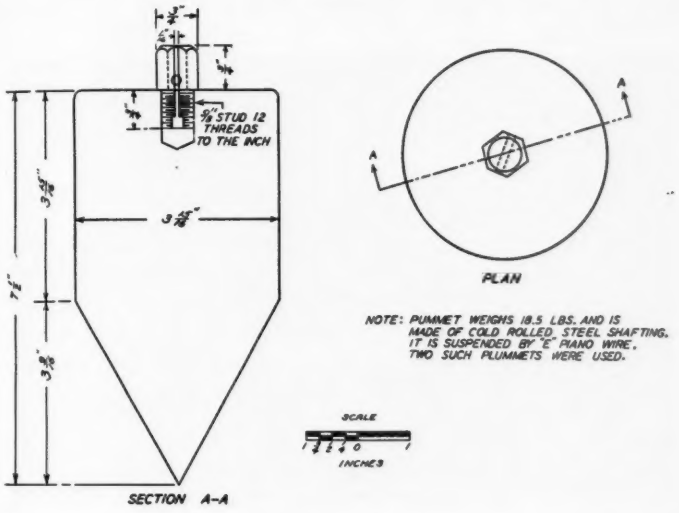
⅝" STUD 12 THREADS TO THE INCH
SECTION A-A
A
A
PLAN
NOTE: PLUMMET WEIGHS 18.5 LBS. AND IS MADE OF COLD ROLLED STEEL SHAFTING. IT IS SUSPENDED BY "E" PIANO WIRE. TWO SUCH PLUMMETS WERE USED.
SCALE
INCHES

FIG. 5.—SHAFT PLUMMET.

the alignment and grades had to be carried down the shaft into this heading.

The elevation was carried to a sharp-edged board fastened on a heavy cross timber over the shaft. From this point a tape was suspended with a weight at its lower end to make it taut. With the level set at the bottom of the shaft, the control for the grade was established.

Two 18-lb. plummets (Fig. 5), were suspended with piano wires from the top into buckets filled with heavy oil at the bottom of the shaft. By trial the transit was set on line and the alignment carried into the heading. The transfer of the lines and grade down the shaft into the heading was done often enough and checked independently until satisfactory control was established. About 2 400 ft. of tunnel were driven with this control. The taping and leveling in the tunnel were carried to 0.001 ft. The error in the meeting of the headings was only 0.1 ft. in both alignment and grade.

For trimming the tunnel and also for the control of pouring the invert,

side walls and the arch, a plug was set in on each side of the springing line every 25 ft., in addition to the plugs set into the crown every 100 ft. during the excavation. Into these plugs were driven nails with their heads exactly on the minimum concrete line and their centers on the springing line for that particular station.

Shaft and Tunnel Excavation.

The shaft was sunk through solid rock, except for about 14 ft. of earth cover at the top. About 24 sump holes and 84 other down holes were put in from 6 to 8 ft. deep, using 4 jackhamers. The position of the sump holes

Fig. 6.—East Portal Timber Protection.

was shifted from one side to the other. About 2 lb. of 40-per cent. DuPont special gelatin dynamite was used per cubic yard of rock excavation. The sump holes were shot direct and the others with up to 5 delays. Muck was loaded by hand into 1-yd. steel buckets and hoisted up to the surface by a 15-ton Northwest crane. The shaft had about 31 cu. yd. excavation per foot, and the general average progress was 1.3 ft. per working day.

Well outside of the concrete section of the shaft, permanent timber cribbing 12-in. round, was put in near the surface through the earth section of the shaft. When the shaft was completed a head frame and guides were built and a one-car cage installed for hoisting the muck from the tunnel to the surface.

At the entrance of the east portal, the drift was made through soft ground for about 90 ft. with timber sheathing all around (Fig. 6). From here on the entire excavation was through solid rock, mica schist with inter-

mittent narrow veins of gneiss. Occasionally, however, veins of Williamsburg granite were encountered. Mica schist and gneiss were easy to drill, but shooting was very hard. The cut holes sometimes were shot up to 3 times. Williamsburg granite was very hard to drill, sometimes a bit drilling not more than two feet, but it shot very easily.

The entire tunnel was driven with full face heading, except occasionally. When the mucking of the previous shot was delayed, heading and bench methods were used for short distances. As soon as a shot was made, a cross

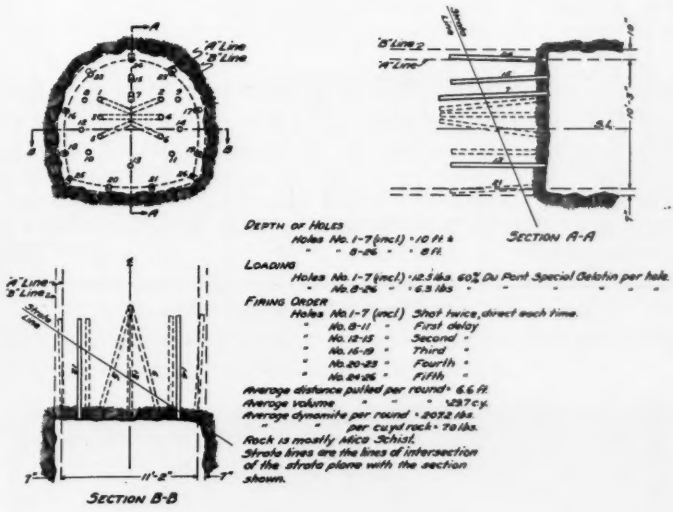
A
A
"A" Line
"B" Line
B
B
"B" Line
"A" Line
Section A-A
Depth of Holes
Holes No. 1-7 (incl) - 10 ft
" " 8-26 " - 8 ft.
Loading
Holes No. 1-7 (incl) - 12.5 lbs. 60% Du Pont Special Gelatin per hole.
" No. 8-26 " - 6.3 lbs " " " " "
Firing Order
Holes No. 1-7 (incl) Shot twice, direct each time.
" No. 8-11 " First delay
" No. 12-15 " Second "
" No. 16-19 " Third "
" No. 20-23 " Fourth "
" No. 24-26 " Fifth "
Average distance pulled per round - 6.6 ft.
Average volume " " " - 1297 c.y.
Average dynamite per round - 2032 lbs.
" " per cu.yd rock - .70 lbs.
Rock is mostly Mica Schist.
Strata lines are the lines of intersection of the strata plane with the section shown.
"A" Line
"B" Line
Strata lines
11'-2"
Section B-B

Fig. 7.— Method of Drilling and Shooting Heading of Tunnel.

bar was set up and 3 Leyners were mounted on it. While the mucking was being done, the drilling crew drilled the upper half of the heading standing on the muck pile and the remaining holes were drilled as soon as the mucking was completed. The total number of holes in the heading varied from 25 to 32; most of the tunnel was driven with 26 holes consisting of 7 cut holes, 8 relievers and 11 square ups (Fig. 7). The cut holes were put in as a rule right about the center, but they were moved anywhere depending on the location of the seams in the rock. DuPont 40- and 60-per cent. special gelatine dynamite was used with instantaneous and 5-period delay electric blasting caps. The average draw was about 7 ft. per round, and the average dynamite consumption, exclusive of the tunnel trimming, was about 7 lb. per cu. yd. of excavation. Steady progress was maintained averaging per working day 12.3 ft. at the east and 10.4 ft. at the west heading (Fig. 8). The drilling crew did the loading and shooting.

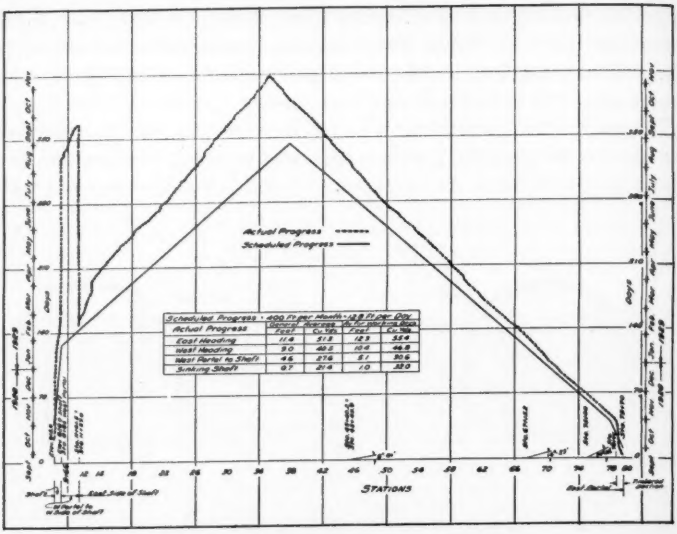

FIG. 8.— PROGRESS CHART OF TUNNEL.

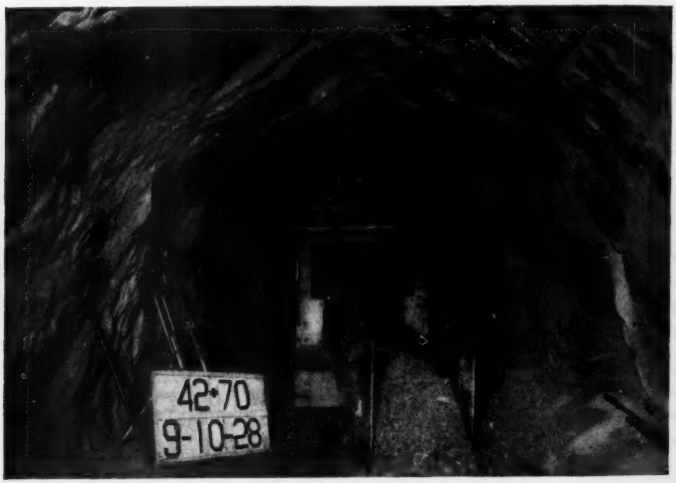

FIG. 9.— MUCKING MACHINE IN TUNNEL.

All dynamite sticks were split and tamped with from 4 to 12 cartridges to the hole; as a rule, when space permitted, as many tamping bags were used in the same hole. Occasionally the hole was filled up with dynamite without any tamping bags.

A small electric-driven Conway mucking machine (Fig. 9) was used at each heading for loading the muck into 1-yd. wooden end-dump cars, which were hauled out in 4- to 6-car trains with an electric dinkey at the east heading over a narrow gauge track, and the muck was used for building a road of about 1 200 ft. outside the portal. The muck at the west heading was hauled by mules, one car at a time, to the bottom of the shaft, where in a 6 ft. by 6 ft. steel cage it was hoisted up to the surface through the shaft and put in spoil banks.

No artificial drainage was necessary at the east heading, as the tunnel here was driven up grade. At the west heading, however, even though the tunnel was comparatively dry, one Worthington, air-driven, 6-in. by 4-in. by 6-in. pump was used to pump the water from the heading into a sump hole at the bottom of the shaft, where an electric-driven Buffalo centrifugal, 3-in. by 2-in. pump was used to pump the water through the shaft to the surface.

The tunnel was ventilated with 4 Coppus Vano 3 200-r.p.m. blowers and with 12-in. diameter canvas ventilating tubing 6 000 ft. long. It took about 10 minutes for ventilation of the tunnel after the cut holes were shot, and 1 hour and 30 minutes after shooting the heading.

There were two 11½-hour shifts per day. The scheduled progress was 400 ft. per month per heading, or about 13 ft. a day. A bonus system was established for the drilling and mucking crews. For every foot extra over the specified distance of tunnel excavation per week the foreman received a bonus of $1.00, the drillers and mucking-machine operator 70 cents and the dummies and muckers 35 cents. At the east portal the specified distance was 98 ft. per 7-day week during the first 2 300 ft. of excavation, 91 ft. during 1 600 ft. of additional excavation and 84 ft. for the remaining 564 ft. of excavation.

Great care was exercised in the location, drilling, and shooting of the square-up holes so as not to have any overbreakage. Considerable time was spent on trimming, with an average cost of 13 cents per square foot area or $5.10 per lineal foot of tunnel. Tunnel excavation per foot was 4.50 cu. yd. at $10 per cu. yd. The trimming cost was more than 13 per cent. of the total excavation. The average section was 1½-in. within the payment line, and it so happened that the value of the excavation and concrete saved per foot was exactly equal to the cost of trimming.

No rock tunnel should be driven too large in the first place so as to avoid trimming later on. Even without the trimming cost, the tunnel would be rather expensive, not only for the extra rock excavation but also for the necessary additional concrete to be put into the overbreaks. There is a limit also as to how tight the bore should be driven in the first place in order

to avoid overbreakage. If a tunnel is driven too tight the trimming expense may amount to more than the value of the anticipated economy in rock excavation and concreting. It is good practice to attempt to remain within the payment lines so that the trimming cost will be offset by the saving in excavation and concrete.

The contractor had 2 Ingersoll-Rand stationary electric air compressors, one of 900 and the other of 600 cu. ft. per min. capacity, near the east portal. This plant supplied the air necessary for the drills, pumps, and the concrete guns. The air to the west heading was supplied by a pipe line from 2 to 8 in. in diameter and about 10 000 ft. long with a 200-cu. ft. steel tank air receiver near the heading. The line was carried over 1½ miles of rough ground and then down the shaft into the heading. On account of cold weather the entire pipe line was given about 3 ft. of earth cover. There were about 25 sags in the line with drain cocks at the lowest points. These low spots were exposed, and during freezing weather constant fires were provided to keep them free.

The steel was sharpened with a Sullivan, S-55 drill sharpener, using a coal furnace.

The drilling and trimming crews per shift consisted of 1 foreman, 3 drillers, and 3 dummies or helpers. Mucking crews were made up for each heading as follows:

	East Heading	West Heading
Mucking machine operators	1	1
Muckers	4	4
Electric dinkey operators	1	..
Brakeman	1	..
Dumpmen	1	3
Mule skinners	..	2
Cagemen	..	1
Hoisting-engine operators	..	1

During January 1929, when the heading had advanced about 2 000 ft. at the east portal, observations were made for a few days and the following data obtained:

Actual drilling done during 11½-hour shift, 8 hours and 7 minutes. Drilling at the heading 29 ft. per drill per hour. It took:

48 minutes to load 26 holes in heading and to shoot the cut.
38 minutes to reload the cut, wire and shoot the heading.
18 direct and 17 delays per shot.
3 minutes to load 1-cu. yd. car by mucking machine.
4 minutes to switch 3 cars by 5 men.
1½ minutes to dump 1-cu. yd. car by 2 men.
50 minutes to lay one 15-ft. section of track by 5 men.

The mucking machine was idle 4 minutes between loading successive cars.

The work was shut down 10 per cent. of the time due to breakdown and repairs of the equipment.

A very careful analysis of actual costs of excavation, made in January, 1929, showed the following, not including overhead or profit to the contractor.

ACTUAL COST PER CUBIC YARD EXCAVATION.

Item.	Tunnel.		Shaft.	
	$	%	$	%
Labor	6.43	69.6	6.21	71.6
Explosives	1.09	11.8	0.55	6.3
Power	0.49	5.4	0.37	4.3
Fuel	0.07	0.7	0.19	2.2
Equipment	1.16	12.5	1.35	15.6
Total	9.24	100.0	8.67	100.0

For the labor item the wages per hour were: Foreman $1.10; hoisting engineer $1.00; drillers $0.70; mucking machine operator $0.75; electric dinkey operator $0.60; unskilled laborer $0.50; blacksmith $0.90; helper $0.60; compressor man $0.70; and carpenter $1.00.

CONCRETE.

The mix for the concrete was 1 bbl. cement weighing 376 lb., 9 cu. ft. sand measured loosely, and 16 cu. ft. ballast measured loosely. The cement, in addition to conforming to the specifications of the American Society for Testing Materials, was required to have less than 1½ per cent. SO_3 content. The sand was to be of well graded, hard durable stock all passing, when dry, a screen having ¼-in. diameter holes and not more than 30 per cent. by weight passing a sieve having 50 meshes per linear inch. The ballast was required to be of well graded, hard, durable stock with no particle larger than that which would pass through a 2½-in. diameter ring and with all particles smaller than ¼-in. removed. The mixing time was three minutes per batch. An average of 6½ gallons of water was used per bag of cement.

The tunnel muck was not suitable for concrete aggregates. The contractor installed a Farrell crushing plant with a rotary screen in a gravel borrow pit near the east portal. The borrow pit was a glacial deposit containing many hard cobbles, which when crushed provided the coarse aggregate. There was an abundance of sand, in fact about 50 per cent. of the sand that came through the hoppers had to be wasted. The borrow pit area was stripped to a maximum depth of about 4 ft. and the aggregates, without being washed, were clean enough to be used in the concrete. A 15-ton Northwest crane, with ¾-cu. yd. clamshell bucket, did the excavation, casting the material several times when necessary, and piled the material onto a platform, where about 3 men fed the jaw of the crusher by shoveling by hand. They also picked out mica schist stones found in the material.

The crushing plant worked two 10-hour shifts per day, each crew con-

sisting of 1 foreman, 1 crane operator, and about 8 laborers. The average costs per cubic yard to the contractor were about $1.50 for gravel, and $1.00 for sand.

For pouring the concrete from the east portal, the mixing plant was located near the portal. The concrete was mixed in a 4-bag batch, electric operated Ransome mixer, and discharged directly into 1 cu. yd. end-dump wooden cars. Trains of up to 4 cars were hauled by small Plymouth gas and Mancha electric dinkeys over the narrow-gage track to the place of pouring.

The invert was poured first, then the side walls to the springing line, and then the arch. During the pouring of the invert, the night crew thoroughly cleaned the bottom of the tunnel to solid ledge for a sufficient

FIG. 11.— ARCH FORMS IN PLACE.

distance ahead for the concreting crew during the next day. No backfilling with muck, even in large overbreaks, was allowed. About 6 000 ft. of invert were poured from the east portal concrete plant. The general average progress per shift for the invert was about 110 ft., with approximately 35 cu. yd. of concrete, although as many as 240 ft. of invert per shift were poured.

About 5 000 ft. of side walls and arch were poured from the concrete plant at the east portal. This concrete work was started at two points about 2 200 ft. apart, progressing towards the east portal from both places. Collapsible, steel-lined forms, both for the side walls and the arch were made on the job in 50-ft. sections. The side walls were poured always three or four sections ahead of the arch (Figs. 10 and 11).

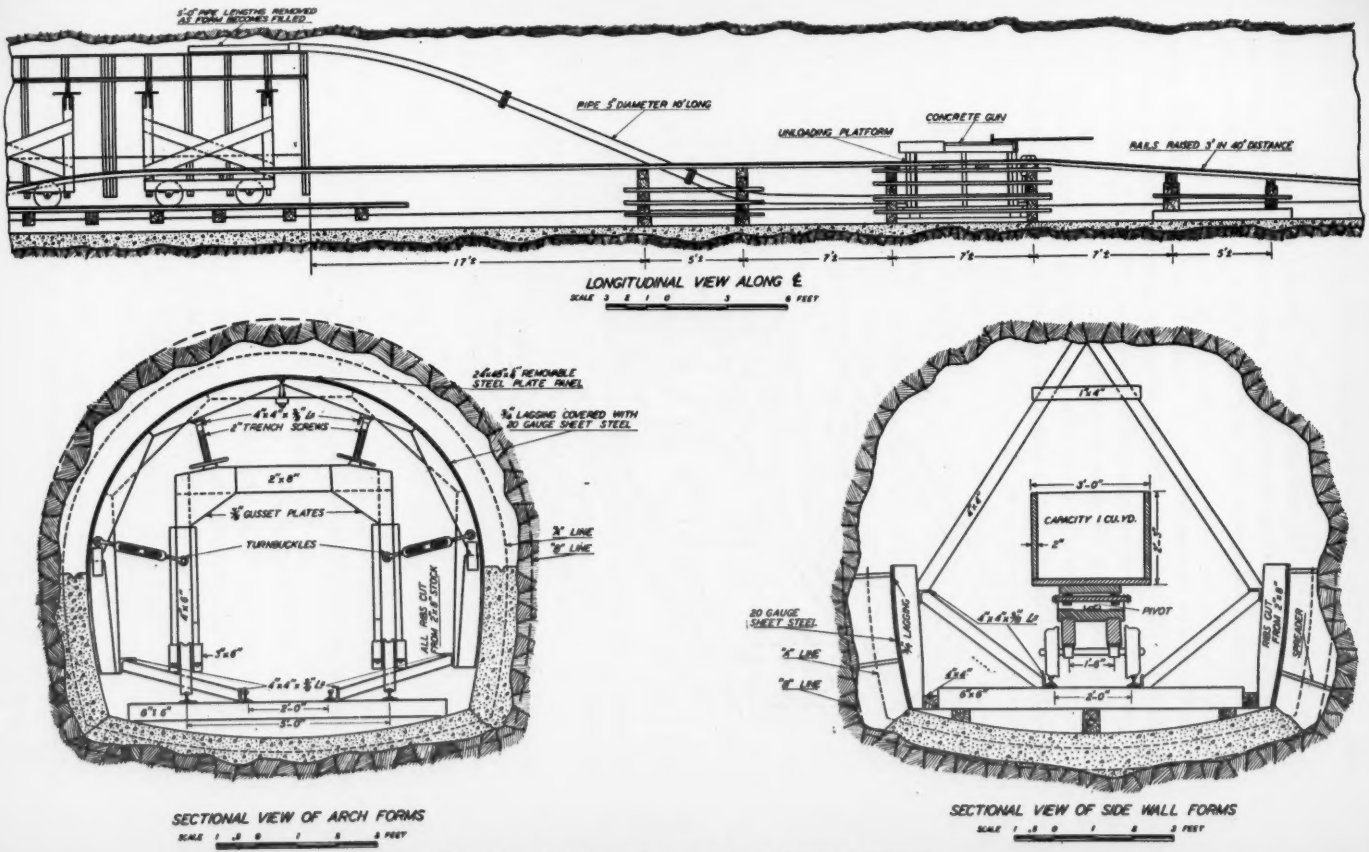

FIG. 10.— CONCRETE LINING FORMS USED IN TUNNEL.

The concrete for the side walls was shoveled from the cars by hand onto chutes leading back of the forms. The arch was shot by pneumatic process with about 70 lb. per sq. in. pressure, using a ⅓-cu. yd. concrete gun (Fig. 12).

In the mix of the concrete for the arch, the contractor, without extra charge, added 0.7 bags of cement per cubic yard of concrete so as to have a more workable concrete than could be obtained with the specified normal mix without increasing the cement-water ratio.

A crew consisting of one foreman and eight laborers was continuously kept busy with the forms, stripping, moving, and re-setting. The concrete

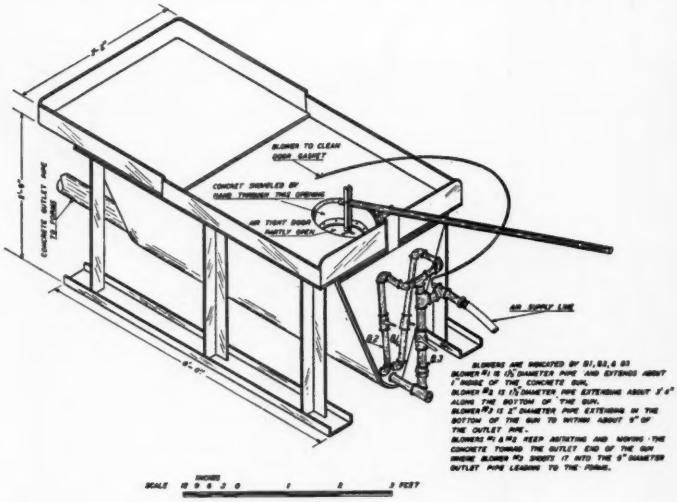
BLOWER TO CLEAN DOOR GASKET
CONCRETE SHOVELED BY HAND THROUGH THIS OPENING
AIR TIGHT DOOR HASTILY OPEN
AIR SUPPLY LINE
#2
#3
BLOWERS ARE INDICATED BY #1, #2, & #3
BLOWER #1 IS 1½" DIAMETER PIPE AND EXTENDS ABOUT 1' INSIDE OF THE CONCRETE GUN.
BLOWER #2 IS 1½" DIAMETER PIPE EXTENDING ABOUT 3' 6" ALONG THE BOTTOM OF THE GUN.
BLOWER #3 IS 2" DIAMETER PIPE EXTENDING ON THE BOTTOM OF THE GUN TO WITHIN ABOUT 9" OF THE OUTLET PIPE.
BLOWERS #1 & #3 KEEP AGITATING AND MOVING THE CONCRETE TOWARD THE OUTLET END OF THE GUN WHERE BLOWER #2 SHOOTS IT INTO THE 6" DIAMETER OUTLET PIPE LEADING TO THE FORMS.
SCALE
FEET

FIG. 12.— CONCRETE GUN OF ⅓-CU. YD. CAPACITY.

crew consisted of one foreman, one concrete gun operator, and about six laborers.

Each concreting crew, during two shifts, poured a 50-ft. section of side forms and arch, thus pouring about 100 ft. of lining, exclusive of invert, per day.

It took, on an average:

3½ hours to pour 50 ft. of side walls, 17 cu. yd.
5 hours to strip, clean, oil, and reset 50 ft. of side forms.
7 hours to pour 50 ft. of arch form, 35 cu. yd.
4 hours to strip, clean, oil, and reset 50 ft. of arch forms.
10 hours before stripping the side wall forms.
10 to 12 hours before stripping the arch forms.

A second mixing plant was located at the top of the shaft for concreting the lining of the shaft and the tunnel from the west portal, Station 8+85

to Station 26+00. A two-bag batch, electric driven, Ransome mixer was used. The concrete was dumped into a rectangular open chute leading into a 6-in. diameter pipe down the shaft to the bottom. Here the concrete was discharged directly into 1-cu. yd. cars and pushed by men a maximum distance of about 1 500 ft.

It was soon found that the two-bag batch mixer did not have sufficient capacity, and that the 6-in. diameter pipe chute down the shaft was too small, plugging quite often. They were replaced by a similar type four-bag batch mixer and an 8-in. pipe chute.

The main shaft and the air-vent shaft, including the landings, were poured in lifts ranging from 5 to 10 ft., with 66 to 220 cu. yd. respectively. Construction joints, as a rule, coincided with the elevation of the successive rack support frames. The concrete was placed directly from the chutes, a length of chute pipe being taken off as the concrete elevation came up higher in the shaft. Special care was taken to make the construction joints watertight in the partition wall between the main shaft and the air-vent shaft. For this purpose ¼-in. thick, 4-in. high, and 9 ft by 10 ft. welded rectangular steel bands were used.

The contractor was permitted to use sand bags for bulk heads between the forms and the rock at the end of each section of the forms. He was, however, required to put in additional 2-in by 4-in. keys. This method of bulk-heading is very satisfactory, and its cost is negligible compared with the wooden plank bulk heads that were used for the diversion tunnel sections.

It was almost impossible to hold the forms true to line, due perhaps to not enough bracing, and especially to the vibration caused by the concrete trains going through the jumbos of one set of forms.

The job was good except that the construction joints at the springing line always had an offset, sometimes up to ¾-in. This offset had to be chipped high enough, repaired and finished in order to produce the effect of a curved surface. This required the constant time of two finishers. For construction joints 1:2 mortar was used. For patching up and for repairs a mix of one part of sand and one part of cement was used. In some cases, when water seeped through the spot, the patch would not hold, and a mix of water glass with neat cement was used for quick setting and satisfactory finish. The water-glass mix took about one-half minute for setting completely.

Briquettes were made daily in the field, and a complete record was kept. The average strength of all the field briquettes was 262 lb. per sq. in. for 7 days and 357 lb. per sq. in for 28 days; a few cylinders made for compression tests, giving values from 2 690 to 3 530 lb. per sq. in. for 28 days. The average ratio between the tensile and compressive strengths was found to be 9.2.

Tunnel Pantograph.

A new tunnel pantograph (Fig. 13) was designed and made on the job for a ratio of 30 to 1, consisting mainly of the following parts: (1) the base of 1½-in. in diameter and 2 ft.-8-in. long steel pipe supported by a plate and three 1-in. diameter legs, each leg being held by ½-in. diameter pipe for

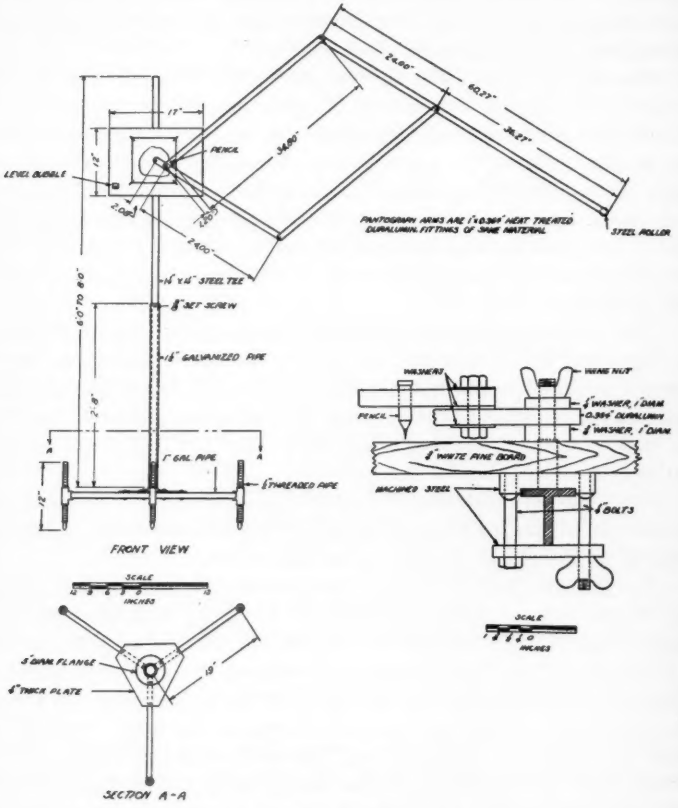

Fig. 13.— Tunnel Pantograph.

leveling the instrument; (2) the stem of 1¼-in by 1¼-in. steel "T" section 6 ft. long, held firmly inside the base at any desired elevation; (3) the board of 1-in. thick white pine, 12 in. by 17 in., with necessary fittings and a level bubble, held on the stem at any height; (4) the pantograph arm and fittings, made of 1 in. by 0.364 in. heat-treated duralumin, supported on the center of the board.

Pantograph sections were taken of the bore every 25 ft., and more often

when necessary, throughout the entire length of the tunnel in order to determine the exact quantity of the excavation and also the shape of the sections. These data were used also in computing the actual concrete placed in lining compared with payment quantities.

The required minimum excavation and thickness of concrete lining, and also payment lines are shown on the typical section (Fig. 4).

COMPARISON OF ACTUAL AND PAID QUANTITIES PER LINEAL FOOT OF TUNNEL.

	Cu. Yd	Per Cent.
Excavation:		
Minimum ("A" line)	3.57	...
Payment ("B" line)	4.50	...
Actual	4.31	...
Payment less actual quantities	0.19	4.4
Concrete:		
Minimum ("A" line)	0.645	...
Payment ("B" line)	1.57	...
Actual	1.38	...
Payment less actual	0.19	12.1

GROUTING.

Before the lining was started a careful survey was made of the stations of running water, large seams in the rock, and also the mud pockets. Drain pipes were placed through the forms to take care of the running water while pouring the concrete. Later on these pipes were grouted. The purpose of grouting was to fill up the space left between the concrete lining of the tunnel and the rock. Grout holes were put in the center of the roof of the tunnel every 20 ft. Every fourth hole was grouted first. After allowing 48 hours for the grout to set, the holes half way between, that is 40 ft. center to center, were grouted. These also were allowed 48 hours to set. Then the remaining holes were grouted.

The first set of holes was grouted starting with about 10 lb. per sq. in. pressure and increasing it up to about 50 lb. per sq. in., or until grout started flowing freely in the neighboring holes. The same procedure was followed with the half-way holes, except that the pressure was increased up to 100 lb. per sq. in. in case no grout appeared in some other holes before then. The final holes were grouted in all cases up to 100 lb. per sq. in. pressure. In the final holes, however, in case the water, if any, did not stop coming through the grout hole with a pressure of 100 lb. per sq. in., the grout pressure was increased up to a maximum of 150 lb. per sq. in.

The amount of grout taken in any hole back of the lining in a mica schist tunnel is no criterion of the void space between the concrete and the rock in a section, say 10 to 15 ft. away. The assumption was that if the tunnel were thoroughly grouted every 20 ft., all the voids would be filled, and that the amount of grout taken in the last holes would indicate whether or not additional intermediate holes were to be put in. In many sections of the tunnel the adjacent holes took considerable grout, and yet intermediate

holes took as much. On the other hand, in some sections of the tunnel the 20-ft. distant holes took no grout, and yet intermediate holes did take some grout. It was concluded that the grout was stopped by uneven breaks or juts of the rock in passing from one hole to another at 20-ft. distances. Undoubtedly, in some cases, grout did travel a great deal more than 20 ft. as evidenced by the grout seeping through the concrete lining at some farther point where the lining was honeycombed. The water, however, seemed to travel farther. As grouting continued in the nearby locations, where apparently the concrete was in first-class condition, the water started seeping through. The real problem was to determine whether all the void space between the concrete and the rock was filled up with grout. This could not be ascertained by the quantity or the lack of grout taken in any hole. This problem was solved by the help of the pantograph sections, from which the necessary yardage of concrete was computed for different sections. By knowing the exact quantity of concrete actually placed within those sections from careful records, it was possible to determine how much space was left in that particular section to be filled up with grout. As the water came through at different places in the lining after the grouting in the crown was completed, additional grout holes were put in those wet spots. There were 103 such holes, about 50 per cent. taking grout, some of which took as many as 77 bags.

The grout pipes were 1½ in. in diameter and 4 in. long with a 2-in. sleeve at the bottom (Fig. 14). One thickness of a 2-in. wide strip of cement bag dipped in grout was wound around the pipe and forced into the hole of the concrete lining. To the sleeve at the bottom of the grout pipe was attached a valve which was connected to the grout hose of the grouting machine. When the grouting was completed for any hole, the valve at the bottom of the sleeve was closed before the grouting hose was disconnected. The grout in the hose was discharged back into the grout chamber through a bypass near the grout machine. Thus no grout was wasted, and no time was lost in cleaning out the grout hose.

Two Worthington pattern, duplex piston, 6 in. by 4 in. by 6 in., air-driven pumps were used as grouting machines (Fig. 14). They were mounted on flat cars and hauled over the narrow-gage track in the tunnel to the places of grouting. On each flat car was fitted a mixing chamber 2 ft. by 4 ft. by 3 ft. The grout batches consisted of 2 bags of cement and about 13 gal. of water, which were mixed by hand for about 3 minutes. The mix was also agitated by going through the pump into the mixing chamber over and over again.

After the grout was set, the grout pipe and the valve were removed and cleaned thoroughly to be used over again. About 4-in. of opening in the concrete lining below the grout were filled up with concrete and pointed.

The contractor had the option, in holes taking more than 1 bag of cement, to use grout mix of 1 part cement and 2 parts of clean fine sand. However, he decided on using entirely neat cement with about 1 to 1 mix. With a neat cement mix, the grout will travel farther and creep in places

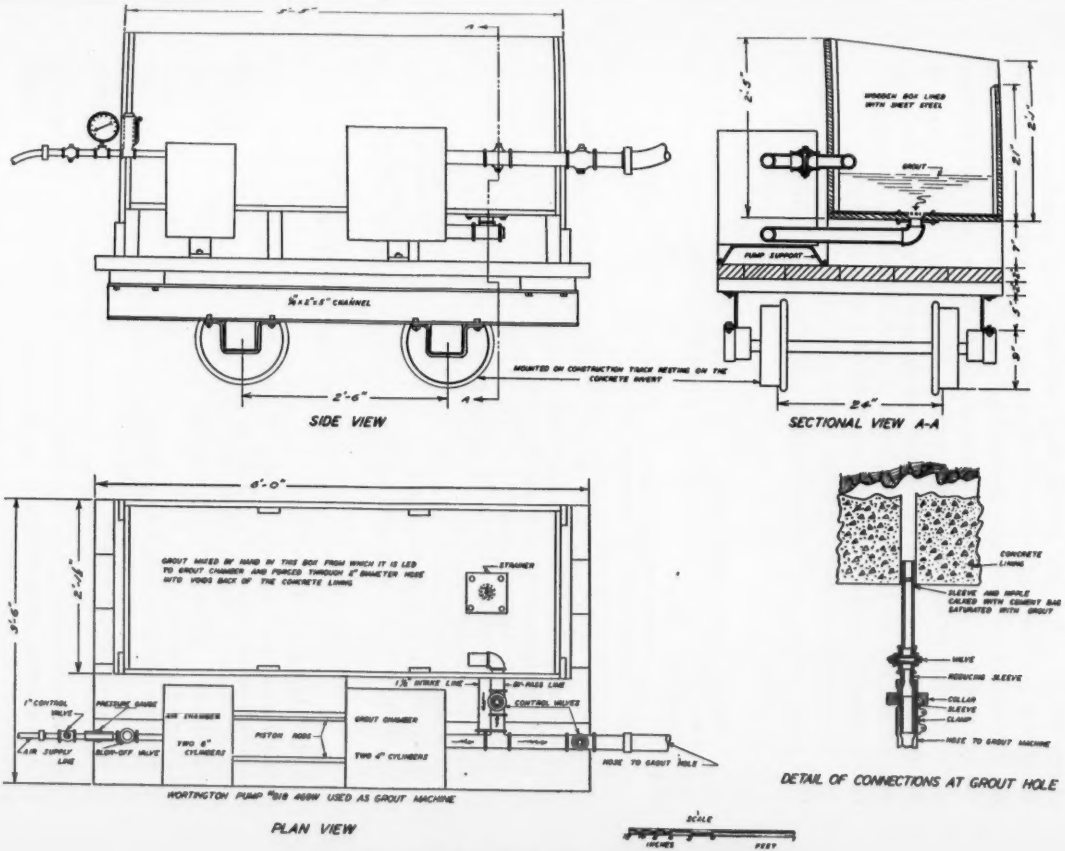

FIG. 14.— CONTRACTOR'S GROUTING EQUIPMENT.

more easily filling the honeycombs and voids in the concrete than the grout and sand mixture. In the case of grouts containing 2 parts sand and 1 part of cement, often the sand, due to heavier weight, will stop during the discharge of the grout and let the fluid travel farther, thus forming sand pockets which eventually will stop the passage of the grout coming in later on. It may look as if the neat cement grouting will be more expensive than the one mixed with sand. Taking into consideration the extra wear and tear and more often repairs of the pumps due to sand, the possibility of putting in additional grout holes on account of the previously mentioned sand pockets and also the time element, it will be cheaper in the end to use a neat cement mix even though a few more barrels of cement may be required.

There were two grouting gangs working two 11½-hour shifts per day, each group consisting of 1 grout-machine operator and 2 laborers.

Grouting Data:

Number of grout holes in crown	336	
Grout pipes set in concrete, due to running water	15	
Special grout holes	103	
Total number of grout holes	——	454
Yield of grout per bbl. of cement — cu. yd. (1 bag cement with 6½ gallons water.)		0.2
Total quantity of grout used (1560 bbl.) — cu. yd.		315.7
Average quantity of grout used per lineal foot of tunnel—cu. ft.		1.3
Average quantity of cement per grout hole — bags		13.7
Maximum quantity of cement used in grout hole — bags		237
Total yardage of grout to total yardage of concrete in lining — per cent		3.6
Average cost per grout hole		$24.64
Grouting cost per foot of tunnel		1.75
Grouting cost per cubic yard of grout		35.40

TOTAL COST OF POWER TUNNEL AND GATE HOUSE.

Tunnel, Including Open Cut at Portals.

1. Earth excavation, open cut — 7 741.9 cu. yd. @ $2.00	$15 483.80
2. Rock excavation, open cut — 8 519.9 cu. yd. @ $5.00	42 599.50
3. Rock excavation, tunnel — 32 488.6 cu. yd. @ $10.00	324 886.00
4. Timbering — 14 662 M F.B.M. @ $50.00	733.10
5. Cleaning tunnel bottom	3 136.00
6. Grouting, all types — 454 holes @ 24.64	11 185.42
7. Concrete lining — 10 621.0 cu. yd. @ $18.00	191 178.00
8. Bonus — 37 days @ $100.00	3 700.00
Total cost of tunnel	$592 901.82
Cost of tunnel per lineal foot	$83.70

Shaft, Including Approach Channel.

1. Earth excavation, open cut — 5 154.9 cu. yd. @ $2.00		$10 309.80
2. Rock excavation, open cut — 178.0 cu. yd. @ $5.00		890.00
3. Rock excavation, shaft — 3 000.4 cu. yd. @ $7.00		21 002.80
4. Concrete — 2 547.06 cu. yd. @ $18.00		45 847.08
5. Reinforcing steel — 44 309.0 lb. @ $0.05		2 215.45
6. Structural steel:		
Cost, purchase — per lb. @ 6.24 ct.	$7 197.85	
Installation — per lb. @ 3.00 ct.	3 464.02	
Total cost — 115 467.2 lb. @ 9.24 ct.		10 661.87
7. Gate and Appurtenances:		
Cost, purchase — per lb. @ 23.36 ct.	$18 875.00	
Freight — per lb. @ 0.54 ct.	442.08	
Installation — per lb. @ 5.00 ct.	4 054.44	
Additional cost of concrete, 86.8 cu. yd. @ $14.00	1 215.20	
Makers supervision	526.76	
Total cost — 80 793 lb. @ 31.1 ct.		25 113.48
8. Hand railing, access shaft, in place — 77 ft. @ $1.52		117.07
9. 12-in. diameter bypass valve, oil tanks, hand pump, accessories, in place		528.53
Total cost of shaft		$116 686.08

Gate House.

1. Concrete — 250.0 cu. yd. @ $32.00		$8 000.00
2. Cornice — 116.0 ft. @ $3.58		415.95
3. Sills — 55.0 ft. @ $3.28		180.37
4. Reinforcing steel — 19 220.7 lb. @ $0.05		961.03
5. Structural steel — 4 894.0 lb. @ $0.089		434.43
6. Steel sashes, area — 610.0 sq. ft. @ $1.091		665.35
7. Glass and glazing — 252 lights @ 47.27 ct.		119.10
8. Conduits, electric		128.67
9. Spiral stairway, 26 ft., height		548.97
10. Chimney, 20 in. by 24 in. inside, 2nd floor up, 22 ft. high		477.44
11. Roof:		
Steel truss	$810.00	
Porete slabs	541.75	
Ludowici-Celedon slab shingle tiles	1 451.00	
Total — 1 214 sq. ft. @ $2.31		2 802.75
12. Electrical Wiring		885.04
13. Approach to Gate House		3 315.61
Total Cost of Gate House		$18 934.71
Total cost of Power Tunnel and Gate House		$728 522.61

The cost of engineering, Division Engineer's office only, has been about 5 per cent. of the total cost. This included all the preliminary investigations, locations and plans prior to construction; the lines, grades, inspection, testing laboratory, etc., during the construction; and final estimates and record plans after the completion of the job.

The excavation at the East Portal was started on September 11, 1928 and the tunnel was holed through on November 12, 1929. Concreting started on October 13, 1929 and was finished on August 25, 1930.

The work was done by the Board of Water Commissioners of the City of Springfield, Mass., with E. E. Lochridge as Chief Engineer and Hazen & Everett as Consulting Engineers, and the writer as Division Engineer in charge of the construction. Under the writer's supervision, A. V. Lynn and C. Kay acted as Resident Engineers during day and night shifts respectively, and L. P. Hayes had charge of the concrete aggregates and laboratory. The Frazier-Davis Construction Company of St. Louis, Missouri, were the Contractors with R. J. Burns as General Superintendent.

During the construction the members of the Board of Water Commissioners were: Edwin A. Blodgett, Chairman, Dr. George L. Fenn, and Mayors Fordis C. Parker and Dwight R. Winter, ex-officio; James J. Shea served as member during 1932 and as chairman during 1933 with John M. Noonan as member. The present members are Henry F. Punderson, Chairman, John M. Noonan and Mayor Henry Martens, ex-officio.

BOSTON METROPOLITAN WATER SUPPLY EXTENSION.

BY KARL R. KENNISON.*

[*Read February 14, 1934.*]
[*Corrected to April 21, 1934.*]

The development of the Ware and Swift rivers for the additional water supply of the Boston Metropolitan District was started in 1926 and the construction is now about half completed. It is expected that the entire new development will be ready for use about 1940 or shortly thereafter. Following is a synopsis of the subject matter contained in this paper:

Part I. General description of the water supply development.
Part II. Program of the new development.
Part III. Description of the principal construction items.
Part IV. Costs and miscellaneous data.
Part V. Origin of the name "Quabbin".
Part VI. Bibliography.

PART I

General Description of the Water Supply Development.

This Boston district is the fourth largest metropolitan center in the United States, and has a high concentration of population, Massachusetts having the second densest population of any state in the Union with forty-seven per cent. of its people living on seven per cent. of its area. The water district includes Boston and nineteen other cities and towns with a population of 1 500 000, all located within ten miles of the State House. There are fifteen other cities and towns with a combined population of 400 000 which are potential members of the water district. The District now supplies to its members, in normal times, about 140 m.g.d., the chief source of supply being the Wachusett Reservoir with a drainage area which was originally 119 sq. miles but which has been reduced to 88 sq. miles by legislative grants from time to time to the City of Worcester. On the basis of Worcester's reduced present needs, the Wachusett area now available to the District is practically 109 sq. miles. The District also uses regularly a considerable area at the head waters of the Sudbury River, about 22 sq. miles of which are tributary to the Sudbury Reservoir, one of the principal steps in the line of aqueducts which bring the Wachusett water to the District.

The District maintains the necessary distribution reservoirs, standpipes and pumping stations to deliver the water to its member municipalities, who are assessed the entire fixed and maintenance charges as a part of their annual state tax. The cost is proportioned to the different municipali-

* Designing Engineer, Metropolitan District Water Supply Commission, Boston, Mass.

ties, two-thirds on the basis of water consumption, and one-third on the basis of valuation. The Water Division of the Metropolitan District Commission maintains the systems of supply and distribution and will take over the new works upon their completion by the Metropolitan District Water Supply Commission.

There have already been presented before this Association papers describing the present supply and the investigations leading up to the authorization of the extension of the sources to the Ware and Swift rivers. Notable among these is a paper presented February 14, 1922, by X. H. Goodnough, an Honorary Member of this Association, for many years Chief Engineer of the State Department of Public Health and at the time Chief Engineer of the Joint Board which investigated the future water supply needs of the District and recommended extending the sources of supply westward from Wachusett Reservoir to the Ware, Swift and Millers rivers. Mr. Goodnough was untiring in his efforts to secure legislation which would continue the established policy of obtaining water from unpolluted sources with large storage possibilities, and he deserves much credit for the outcome. The water-supply development was also described in a paper by Chief Engineer Frank E. Winsor, a Past President of this Association, published in the December, 1929, issue of the Proceedings of the American Society of Civil Engineers.

The New Supply. The extension of the supply, now under construction, was authorized by Chapter 375 of the Acts of 1926 and by Chapter 321 of the Acts of 1927. To carry out the work, the Governor appointed three commissioners as the Metropolitan District Water Supply Commission, and the actual development commenced in October, 1926, with their appointment of Mr. Winsor as Chief Engineer. The project as authorized, although practically that recommended by the Joint Board in 1921, includes the development of the Ware and Swift rivers only; it does not cover any taking of the waters of the Millers River. Briefly, the plan contemplates the construction, in the valley of the Swift River, of Quabbin Reservoir which will be 18 miles long and hold 415 billion gallons of water, or over six times as much as Wachusett Reservoir. It will have an area of 39 sq. miles and a shore line about 151 miles long, including that of about 60 islands. The depth of water will be about 100 ft. at the water-supply intake, and 405 of the total 415 billion gallons will be available for the District. Tributary to this reservoir will be 186 sq. miles of the Swift River and 98 sq. miles of the Ware River drainage areas.

To impound the water, two large earth dams will be required, the main dam across the Swift River about 2½ miles south of the village of Enfield, and a dike about 2½ miles to the east across the valley of Beaver Brook. The dam will be about 2 640 ft. long, 170 ft. high above the river bed and at least 280 ft. above the surface of sound ledge, and will contain about 4 000 000 cu. yd. above the original surface. The dike will be about 2 140 ft. long, 135 ft. high above the brook bed and 264 ft. above the surface of

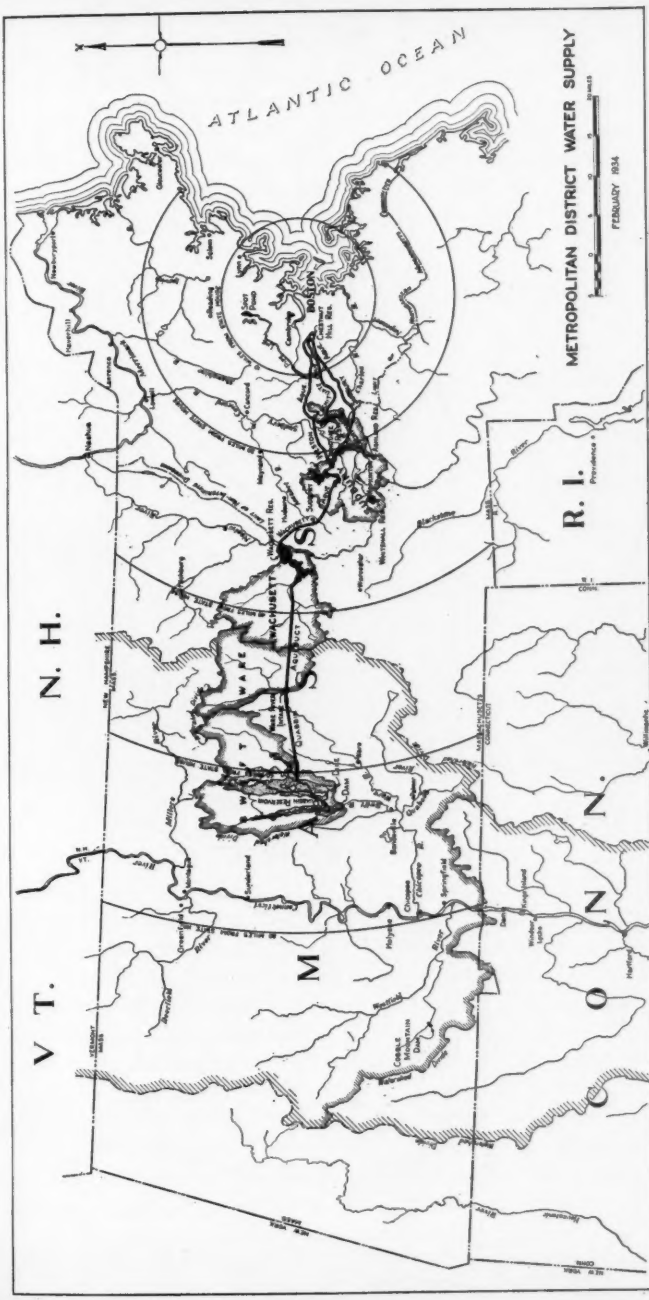

FIG. 1.— METROPOLITAN DISTRICT WATER SUPPLY.

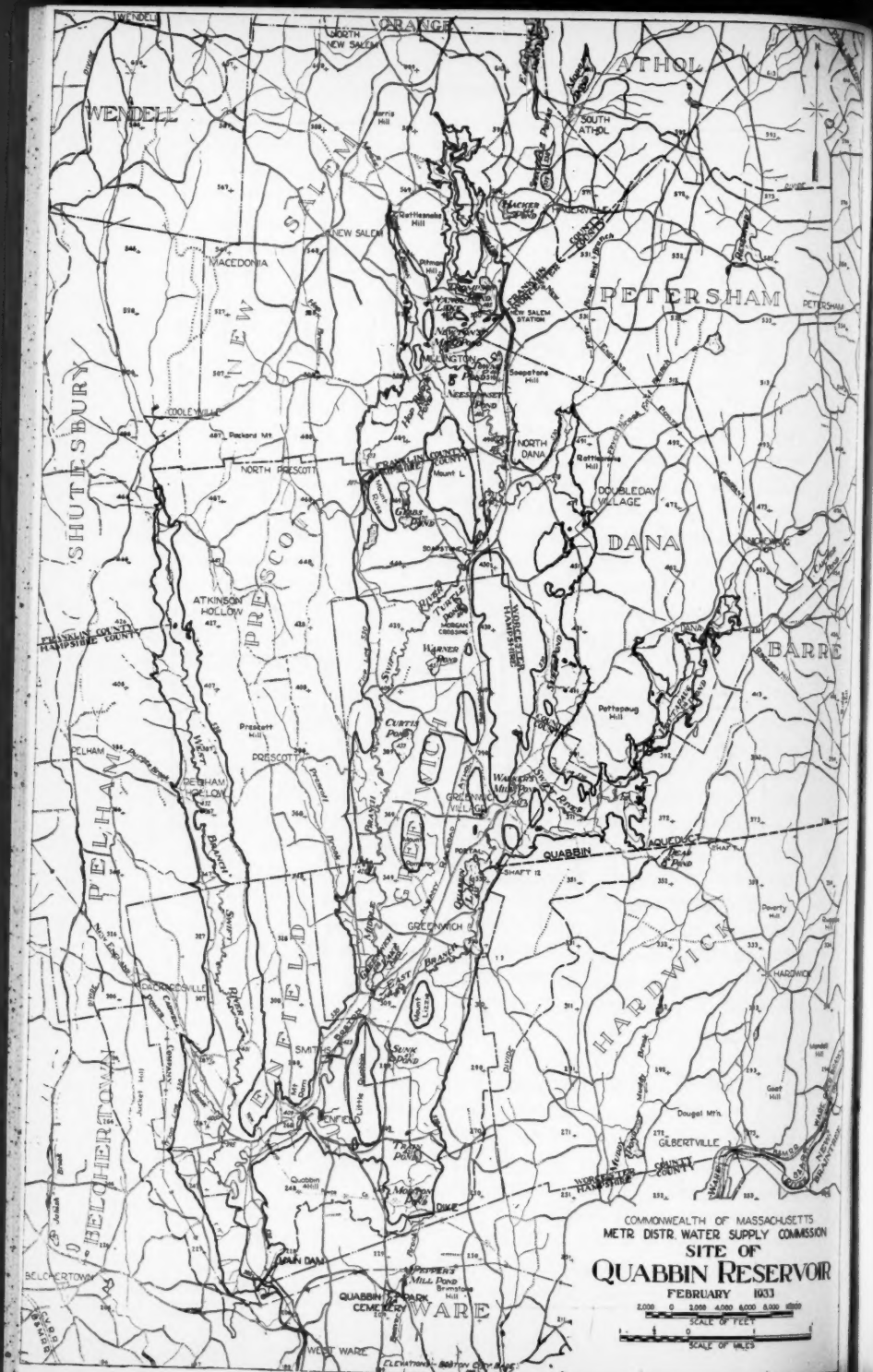

FIG. 2.—QUABBIN RESERVOIR

sound ledge, and will contain about 2 500 000 cu. yd. above the original surface.

The new Quabbin Reservoir will supply Wachusett Reservoir through a tunnel now nearing completion. Unlike any of the existing aqueducts from the Wachusett supply which consist of alternate adjoining sections of tunnel, cut and cover, siphon pipes and some open channel, the new Quabbin Aqueduct is a single tunnel 24.57 miles long between portal face and outlet shaft, making a continuous tube from the intake structure to the outlet shaft 24.61 miles long. This is comparable in length with a similar tunnel 25.1 miles long which the City of San Francisco is now completing through the Coast Range as a part of its Hetch Hetchy Aqueduct. These

FIG. 3.—WARE RIVER INTAKE WORKS AT TOP OF SHAFT 8, A 260-FT. DROP INTO QUABBIN AQUEDUCT TUNNEL. NOVEMBER 27, 1931.

tunnels are the longest in the world and twice as long as the longest railroad tunnel under the Alps. The only other tunnels comparable in length are two water-supply distribution tunnels of the City of New York extending southerly from its Hill View Reservoir in Yonkers, City Tunnel No. 1, 18 miles long and under Manhattan Island itself, City Tunnel No. 2, 20 miles long now under construction through the Bronx, Queens and Brooklyn to a shaft near Governors Island, and the Shandaken Tunnel 18.1 miles long, also a part of New York's water-supply system.

The Quabbin Aqueduct tunnel will tap the bottom of the new reservoir at a point in the town of Greenwich and conduct the waters of Quabbin Reservoir by gravity to the upper end of Wachusett Reservoir in West Boylston. This tunnel will have no gates at the Quabbin Reservoir intake, but its discharge into Wachusett Reservoir will be controlled entirely by

gates at the east or Wachusett Reservoir end. Hence the tunnel will be in effect a part of Quabbin Reservoir itself; the water level will rise in all the shafts and be governed by the level in Quabbin Reservoir, and any water discharged into the tunnel will be free to flow westward back into Quabbin Reservoir.

At about the halfway point the tunnel passes beneath the Ware River at a depth about 260 ft. below the spillway of the Ware River intake works below the village of Coldbrook, so that the Ware River is made tributary to Quabbin Reservoir by the drop into the tunnel at that point. Furthermore, the Ware River intake is high enough so that floods twice as great as any observed in the period of record during the last 20 years can be dropped into the tunnel, and there is sufficient head to force them the entire 11.37 miles westward into Quabbin Reservoir and 13.24 miles eastward into Wachusett Reservoir. In other words, on account of this high level of the Ware River intake works directly above the tunnel it was not necessary to construct any storage reservoir on the Ware River to prevent spillway waste, the Quabbin Reservoir serving to store the run-off from the Ware River drainage area just as effectively as that from its own Swift River drainage area; nor, for the same reason, was it necessary to build the tunnel any larger to deliver the high rates of flood flows from the Ware intake than would in any event be required for the through delivery from Quabbin Reservoir to Wachusett Reservoir.

Towns Flooded by the Reservoir. Several small villages are located in the area to be flooded by Quabbin Reservoir. These will be wiped out in the course of the construction. The area involved, however, is so large that unprecedented measures will be put into effect under the provisions of the legislation which authorizes a realignment of both town and county lines. The towns of Enfield, Greenwich and Prescott will not only be largely flooded but will be wiped out of corporate existence, and portions of each will be annexed to the adjoining towns. After the realignment, the lines will in general follow the water courses so that regardless of the extent to which the reservoir is filled at any time, the dry area of each town will be undivided by the waters of the reservoir. The legislation providing for such drastic changes contains many interesting provisions to protect the rights of individuals as citizens of these towns and counties. It will be several years before this realignment will be made effective; however, the Commission took over the entire administration of the town of Prescott July 25, 1928. The population on the entire watershed of Quabbin Reservoir in 1925, before the work on the new development commenced, was only 21 per sq. mile. Since 1850 there has been a general decrease in the population on the area tributary to the site of Quabbin Reservoir.

The character of the land to be flooded is approximately 27 per cent. arable, 4 per cent. pasture, 5 per cent. swamp and water surface, 21 per cent. pine, 15 per cent. other woodland and 28 per cent. sprout land and mixed growth.

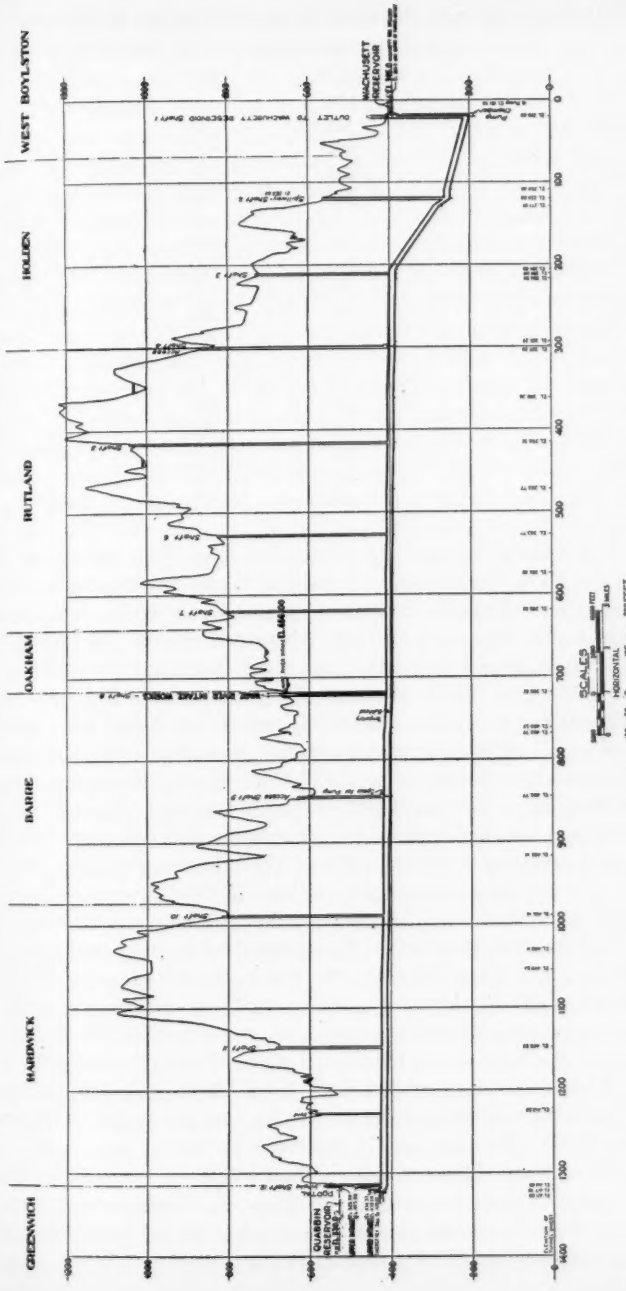

FIG. 4.— PROFILE OF QUABBIN AQUEDUCT.

TABLE 1.— POPULATION OF TOWNS IN QUABBIN RESERVOIR WATERSHED.

Town.	Estimated Population of Town Within the Watershed. Census of 1850.	Census of 1925.	Census of 1930.
Barre	150	150	170
Belchertown	45	50	60
Dana	842	657	505
Enfield	1 036	749	497
Greenwich	838	450	238
Hardwick	70	110	90
New Salem	1 080	444	400
Orange	10	25	25
Pelham	491	260	200
Petersham	1 222	547	550
Phillipston	56	25	25
Prescott	737	230	48
Shutesbury	456	104	100
Wendell	276	132	100
Total	7 309	3 933	3 008

The Watershed Areas and the Connecticut Case. The two rivers, the Ware and the Swift, from which the water is taken, are tributaries of the Connecticut River. Together with the Quaboag River, whose area adjoins them on the south, they unite at Three Rivers to form the Chicopee River which is highly developed industrially and which flows into the Connecticut River just above Springfield. There is no present industrial development on the Ware River above the point of diversion, but below there are, in addition to a mill at Coldbrook whose site was taken for the intake works 9 industrial developments on the 31 miles of river downstream which have developed 187 of the total 330 ft. of drop in the river. These industries will continue to have the benefit of the intervening drainage area tributary to the river, amounting to 121 sq. miles at Three Rivers, in addition to the low flows from the 98 sq. miles from which the flood flows are diverted. On the Swift River there is no important industrial development above the main dam of Quabbin Reservoir. Below the dam only 16 sq. miles contribute their yield at Three Rivers in addition to the 186 sq. miles diverted. Only the industrial development of the Otis Company at Bondsville is located on this stretch of the river, using 43 of the total 75 ft. of drop in the river. On the Chicopee River below Three Rivers 14 industries are affected, which have developed 232 of the total 260 ft. of drop in the river. An additional tributary area of 211 sq. miles is brought in by the Quaboag River. The total tributary area is increased by 88 sq. miles along the course of the Chicopee River, making a total of 720 sq. miles at its mouth before diversion. These facts as to the geographical location and industrial development have been the primary considerations in determining the nature and extent of the development of the new sources.

The only point on that portion of the Connecticut River affected by the diversion at which the natural head is developed for industrial purposes is at Windsor Locks. Here, although there is a potential development of about 50 000 h.p. at 30-ft. head, the present development by the canal on the west bank is unimportant. However, the interest of the War Department in maintaining navigation on the lower Connecticut River made it necessary to obtain from that department permits for the Ware and the Swift diversions. On January 16, 1928, the State of Connecticut filed a bill of complaint with the United States Supreme Court to enjoin the Commonwealth of Massachusetts from making the proposed diversion of waters of the Ware and Swift rivers. The Court's decision on February 24, 1931, denied Connecticut's contention and upheld the right of Massachusetts to make the diversion subject to regulations of the War Department. A paper describing this case was presented to this Association by Chief Engineer Winsor on October 1, 1931, at which time a paper describing a similar case involving an interstate diversion from the Delaware River for the water supply of the City of New York was presented by Thaddeus Merriman.

Yield of the New Supply. By Massachusetts legislation, the District is authorized to take from the Ware River at Coldbrook only the flood flows in excess of 85 m.g.d.; that is, on any day when the natural flow from these 98 sq. miles does not exceed this rate, which is about 1.34 c.f.s. per sq. mile, no diversion can be made. Furthermore, this diversion cannot be made in the dry months between June 1 and December 1 except with the permission of the State Department of Public Health. By Massachusetts legislation, the entire flow from the Swift River area may be diverted except that a sufficient quantity must be released from the reservoir to insure the maintenance of a flow of 20 m.g.d. into the mill pond of the Otis Company at Bondsville located about 5 miles downstream from the main dam. This means that regardless of what the natural flow of the river may be, there must be discharged each day from the reservoir at the main dam a flow of 20 m.g.d. less the yield of the small intermediate drainage area of about 8 sq. miles above Bondsville and below the main dam.

In addition to these restrictions placed upon diversion by Massachusetts legislation, the War Department, in the interest of navigation on the Connecticut River, does not permit any diversion to be made from the Ware River between June 15 and October 15 and requires under certain conditions of drought a release of additional water from Quabbin Reservoir. The necessity of such release is to be determined by the flow in the Connecticut River as measured by the U. S. Geological Survey at Montague, Mass. On any day when the flow of the Connecticut River at this point is less than 4 900 c.f.s., which is about 0.616 c.f.s. per sq. mile, the District must discharge from the Quabbin Reservoir at the main dam 70 c.f.s.; furthermore on any day when the flow is still lower or less than 4 650 c.f.s., which is about 0.585 c.f.s. per sq. mile, the District must discharge 110 c.f.s. These releases are not in addition to those required to guarantee the flow at

Bondsville, but are the total required releases. The restrictions imposed by the War Department reduce the supply available from the Ware and Swift rivers to about 93 per cent. of the maximum that would have been available under Massachusetts legislation, and require approximately 7 billion gallons of storage capacity in which the quantity of water required to be released during periods of drought can be stored over a long period of years.

It is estimated that the additional safe yield, during a prolonged drought, available to the District from these new sources is 210 m.g.d., which means the safe yield of the present sources of good quality in the Wachusett and northern Sudbury areas will be multiplied by about 2¾. It is interesting to note that of this total additional yield, 40 m.g.d. come from the Ware River and 180 from the Swift River, from which must be subtracted a loss of about 25 m.g.d. in evaporation from the large area of water surface in the new reservoir; and about 15 m.g.d. come from a saving of waste over the Wachusett spillway on account of the greater opportunity for economical operation of the two reservoirs in series. This additional yield is being made available in progressive steps, the first being from the Ware which was diverted in 1931 directly into Wachusett Reservoir through the easterly section of the tunnel. However, on account of limited storage available in Wachusett Reservoir, the additional yield obtainable in this way during a critical dry period does not exceed 30 m.g.d., and no large additional yield can be obtained until the tunnel is completed to Quabbin Reservoir.

Emergency Requirements. On account of the shortage in the supply which was threatening at the time the Water Supply Commission was created, this commission was authorized by Chapter 111 of the Acts of 1927 to further develop certain sources of the District in the southern Sudbury area by the construction of pipe lines from Whitehall, Hopkinton and Ashland reservoirs to the reservoirs and aqueducts of the northern Sudbury system. The development of this emergency supply was practically completed late in 1927. It is not an integral part of the new Ware-Swift supply and needs no further comment. Some of the interesting features in connection with this work, including the design of a pipe line located about 26 ft. above the hydraulic gradient, were described in a paper by the author, published in the March, 1933, issue of this JOURNAL.

The level of the Wachusett Reservoir had dropped to 32.3 ft. below its spillway level in January, 1927, and continued approximately 12 ft. lower than in 1926 until in the fall of 1927 heavy rains, unusually heavy for that time of year, not only caused the Vermont flood but insured the filling of Wachusett Reservoir during the next spring. As a result, these pipe lines from the emergency southern Sudbury source were not operated except for test purposes until January, 1930. Thereafter they were used effectively throughout the drought of 1930 and until the completion of construction of the easterly section of the Quabbin Aqueduct tunnel enabled the District to begin the diversion of flood flows from the Ware

River in March, 1931. This was just after the Wachusett Reservoir had dropped to an all-time low of 45 ft. below its spillway level, in February, 1931, and there were only 21 billion gallons left in storage, of which only about one-half was available for draft through the Wachusett Aqueduct. The additional yield obtainable from the Ware River by the completion of this first stage of the development was most timely.

Worcester's Needs. The needs of the city of Worcester for an additional supply of water have been closely tied to those of the Metropolitan District throughout the entire history of the development of these new sources. The Joint Board in 1921 proposed that Worcester be allowed to pump out one of the shafts of the tunnel into her Pine Hill Reservoir drainage area. The report of the special investigating commission in 1926 suggested an even closer association between Worcester and the District in the development of the upper Ware River drainage area. The legislation which created the present water-supply commission authorized the payment of one million dollars by Worcester toward the cost of the District's new supply, in return for which Worcester would be allowed to develop the Quinapoxet Pond area in the upper Wachusett watershed, totaling about 21 sq. miles. Worcester has built a pumping station and a force main from this area into her Kendall Reservoir, but before this was completed she found it necessary, in 1930, on account of the very low level of Wachusett Reservoir, to install emergency pumps and to dam temporarily a small inlet of Wachusett Reservoir in order to increase her facilities for supplementing her supply by pumping from the Wachusett Reservoir.

For the joint benefit of Worcester and the District, the Commission was also authorized by Chapter 262 of the Acts of 1932 to construct a trunk sewer through the towns of Rutland and Holden to a connection with the sewerage system of the City of Worcester for the purpose of maintaining and protecting the purity of the water supplied and to be supplied from the Wachusett Reservoir, the Quinapoxet River and the Ware River.

Legislative Authorization for the New Supply. The various duties of the Metropolitan District Water Supply Commission as outlined in the preceding are specified in certain Acts of the State Legislature. The total amount authorized by the Legislature to be spent for the extension of the water-supply sources of the District to the Ware and Swift rivers, including the southern Sudbury emergency supply, is $65 900 000. Worcester's payment of $1 000 000 was contributed toward this total sum leaving $64 900 000 to be raised by bond issues of the Commonwealth. Bond issues to date aggregate $28 500 000. The enabling acts and the amounts authorized may be summarized on page 158:

The comparative cheapness of this unfailing supply of pure water which is so essential to the very life of the District can be appreciated if we note that the total cost per capita, to the population of the cities and towns in the District, of both the old and the new supply is about 9/10

of a cent a day, and is not expected at any time to exceed a cent a day. This is the wholesale cost which the cities and towns in the District are assessed as a part of the State tax and covers all bond requirements of the old supply, largely financed by 40-year sinking-fund bonds, all bond re-

TABLE 2.— LEGISLATIVE AUTHORIZATION FOR NEW SUPPLY.

No.	Year.	Chapter.	Principal Features.	Amount Authorized.
1.	1926	375	Water Supply Commission, Ware Supply, Purchase of Real Estate for Swift Supply.	$15 000 000
Amended by:				
1-A.	1931	340	Quinapoxet Grant to Worcester.	..
2.	1927	111	Southern Sudbury Emergency Supply.	plus 900 000
3.	1927	321	Swift Supply, Modification of Town and County Lines.	plus 50 000 000
Amended by:				
3-A.	1928	340	Management of Town of Prescott.	..
3-B.	1934	214	Board of Referees.	..
4.	1931	66 (Resolves)	Investigation of Watersheds by Joint Board.	..
5.	1932	262	Rutland-Holden Sewer.	..
			Total Authorized	$65 900 000

quirements of the new supply financed by 30-year serial bonds, and all maintenance costs to the point of delivery to the various municipalities. In addition, the individual consumer pays to his city or town its rates for maintenance and distribution. The old supply and the entire distribution system cost nearly $50 000 000 and is 78 per cent. paid for. For the new supply nearly $30 000 000 have been expended to date, and this supply is about 15 per cent. paid for.

During the District's first 25 years, before construction of the new supply was begun, the assessments on the member cities and towns averaged about 6/10 of a cent per capita per day. The cost of the new supply has increased this gradually to the present cost of about 9/10 of a cent and will maintain it at about this figure during the next ten years, after which it will gradually decrease in another ten years to the cost which prevailed before the present additions were undertaken. Table 3 shows the costs included in the annual District assessment for 1934 compared with those for 1926. The assessments on account of the new supply will be offset to a great extent by the facts that after next year there will be a material reduction of the total bond requirements for the old supply and that by the year 1950 they will have been reduced from $1 754 377 to $145 073.

TABLE 3.— WATER DISTRICT ANNUAL ASSESSMENTS.

	April 1, 1926.	April 1, 1934.
Bond Requirements for the Old Supply		
Payments into Sinking Fund	$174 691	$135 467
Interest on Sinking Fund Bonds	1 393 435	1 393 435
Payments on Serial Bonds	115 000	114 000
Interest on Serial Bonds	147 866	111 475
Total Bond Requirements	$1 830 992	$1 754 377
Miscellaneous Interest Adjustment	12 561	35 000
Additions to the Distribution System	300 000	300 000
Maintenance	769 799	857 624*
Total for the Old Supply	$2 913 352	$2 947 001*
Bond Requirements for the New Supply		
Payments on Serial Bonds		$884 000
Interest on Serial Bonds		898 908
		$1 782 908
Premium on Sale of Bonds		24 330
Total Bond Requirements		$1 758 578
Miscellaneous Interest Adjustment		4 812
Total for the New Supply		$1 763 390

* On the basis of maintenance charges for 1933 which are anticipated to be equalled in 1934.

Personnel. The Commissioner of the Metropolitan District Commission is, *ex officio*, the Chairman of the Metropolitan District Water Supply Commission. The two Associate Commissioners are appointed by the Governor with the advice and consent of his Council to serve for terms of five years. The personnel of the Commission and the engineering departmental heads are as follows:

Commission.

Chairman: Davis B. Keniston, May 28, 1926, to date.
Associate Commissioners: Charles M. Davenport, July 28, 1926, to date.
Joseph H. Soliday, July 28, 1926, to July 29, 1931.
Thomas D. Lavelle, July 29, 1931, to date.
Secretary: R. Nelson Molt, March 3, 1927, to date.

Headquarters Office.

24 School Street, Boston, Mass., October 29, 1926 to December 31, 1930.
20 Somerset Street, Boston, Mass., January 1, 1931, to date.
Chief Engineer: Frank E. Winsor, October 1, 1926, to date.
Designing Engineer and Principal Assistant to the Chief Engineer: Karl R. Kennison, October 16, 1926, to date.
Mechanical Engineer: Walton H. Sears, November 29, 1926, to date.
Consulting Engineers: X. Henry Goodnough, Inc.
J. Waldo Smith (died October 14, 1933).
Chas. T. Main, Inc.
Consulting Geologist: Dr. Charles P. Berkey.

Wachusett-Coldbrook Tunnel Division, Holden, Mass.

Division Engineer: Harold W. Horne, November 1, 1926, to May 23, 1928 (died).
William W. Peabody, July 9, 1928, to date.

Coldbrook-Swift Tunnel Division, Hardwick, Mass.

Division Engineer: Richard R. Bradbury, September 15, 1927, to date.

Quabbin Reservoir Division, Enfield, Mass.

Division Engineer: N. LeRoy Hammond, November 14, 1926, to date.

Southern Sudbury Emergency Supply, South Framingham, Mass.

Assistant Engineer: Walter Knowles, December 20, 1926, to February 29, 1928.

Laboratory

Before Removal to Quabbin Reservoir Division, 175 State Street, Springfield, Mass.
Chemist: Walter H. Junkins, July 23, 1928, to May 1, 1930.

The engineering force, during the last seven years, averaged 125 persons, the maximum, at present, being about 165.

PART II.

Program of the New Development.

It was evident from the start that the item of construction requiring the greatest length of time for completion and subject to the greatest hazards and possible delays was the tunnel. Accordingly, as soon as the engineering force was organized in October, 1926, work was started on the collection of data for preparation of the contract for the east section of the tunnel, the portion extending from Wachusett Reservoir to the Ware River at Coldbrook. The first contract let was for borings with the purpose of locating sound ledge near the surface at suitable sites for shafts from which to carry on the tunnel excavation simultaneously at a number of headings. It was evident that one of these shafts would have to be put down from the high ground in the town of Rutland and that it would have to be 656 ft. deep, considerably deeper than any of the other shafts. Hence a separate contract was prepared as rapidly as possible for sinking this shaft only; this was executed March 30, 1927. It was decided to work the 13.24 miles of tunnel between Wachusett Reservoir and the Ware River at Coldbrook from 8 shafts, No. 1 being at the Wachusett outlet and No. 8 at the Ware River intake, the average distance between shafts being 1.89 miles and the maximum 2.20 miles. To further expedite the work, other shaft contracts were let in advance of the main tunnel contract, one contract for shafts 2, 3 and 4 and another for shafts 6 and 7 being executed on June 1 and 2, 1927, respectively. Shafts 1 and 8, being at the outlet and inlet works respectively, required special designs, and it was not feasible to include them in the other preliminary shaft contracts. An early contract was also executed April 27, 1927, for the construction of a 22 000-volt

electric transmission line along the entire length of the tunnel right-of-way so that power for construction purposes would be immediately available to all contractors. In fact the construction was completed and the line energized in 54 days after the contract was let, so that even the first contractor on the deep Shaft 5 had the benefit of its use.

In the meantime, work was being pushed on the southern Sudbury emergency supply. The entire plan of this development had to be first prepared and submitted to the Legislature; House Document 221, 1927. The Legislature then approved this plan in Chapter 111 as previously noted, and seven contracts were prepared and let and the emergency supply completed ready for use within the year. In the meantime also additional contracts for borings at the sites of the proposed dam and dike for Quabbin Reservoir and for aerial surveys of the reservoir site and along the Ware River were let. Five such contracts for surveys have been executed, one of which, for borings, is still in progress.

The Tunnel Contracts. The first major contract for the tunnel was finally prepared and bids were opened November 18, 1927. It included the sinking of shafts 1 and 8 and the excavation of a heading extending west of Shaft 8 in anticipation of the future extension of the tunnel to the Swift River. This heading was in fact driven 0.95 mile so that the actual length of the Wachusett-Coldbrook Tunnel section, so-called, of Quabbin Aqeduct was 14.19 miles. The bidders were allowed to bid on the easterly half of the work, 6.39 miles, the westerly half, 7.80 miles, or both. Two contracts were awarded. However, both contractors assigned their contracts to one company. Incidentally, a similar option was allowed the bidders on the Coldbrook-Swift section, the subsequent extension of the tunnel, but only one contract was executed. The contract for the Wachusett-Coldbrook section was not executed until April 30, 1928, on account of delays which seemed advisable because of the pending suit brought by the State of Connecticut in the United States Supreme Court. This delay, however, was offset by the fact that the shaft contractors, after turning the headings the required 50 ft. in both directions from the shaft bottoms, were allowed to continue excavating the tunnel headings. By the time the main tunnel contract was let, 1.56 miles of tunnel had been excavated from the 12 headings under the shaft contracts. The tunnel contractors immediately took over the work and continued it with little or no interruption and the excavation was completed ready for lining June 15, 1930.

As the construction of the Wachusett-Coldbrook section of the tunnel neared completion, four contracts were prepared and let for the construction of the Ware River intake works and four for the Wachusett outlet works. These included the construction of the low diversion dam across the Ware River, the construction and equipping of the intake building and the outlet building and the grading of the grounds at these two points. A contract was also let for filling swampy areas upstream from the intake and also one for the construction of a head house at Shaft 4. The construc-

tion was sufficiently completed in January, 1930, to allow the first diversion of the next available flood flows to be made from the Ware River to Wachusett Reservoir, although much of the operating equipment at the intake works was at the time housed in temporary wooden enclosures. The buildings themselves were completed and the grading of the grounds finished late in 1931.

This completed the construction program for the Ware supply except that a contract was executed July 11, 1933, for about 9½ miles of sewer now under construction from the vicinity of Rutland Center through Holden to connect with the Worcester sewerage system, under an agreement with that city. Here also bids on either or both of two portions were invited, but only one contract was let. Some of the interesting features in connection with this work, including the design of a new type of open nozzle, for the measurement of sewage flow, were described in a paper by the author published in the January, 1934, issue of the Journal of the Boston Society of Civil Engineers. These open nozzles were recently tested and rated in the hydraulic laboratory of the Massachusetts Institute of Technology.

The first step in the construction program for the Swift supply was the extension of the tunnel westward to the Swift River to tap Quabbin Reservoir by the construction of the Coldbrook-Swift Tunnel section, so-called, of Quabbin Aqueduct. Since the requirements for speed were not so urgent in the case of this extension west of the Ware intake, the distance between shafts was increased, only four more shafts being required in the 10.38 miles remaining to be excavated. The maximum distance between shafts is 3.17 miles.

The last shaft, No. 12, is located practically at the extreme westerly end only about 320 ft. east of the rock face portal, and at a point where the original surface is 30 ft. below the future flow line of the reservoir. This shaft is designed as a high-level intake to control the draft from the upper 45 ft. of the reservoir also to permit drawing bottom water. The contract includes the construction of about 200 ft. of cut and cover section extending from the rock face portal of the tunnel to a low-level intake in the of the valley. This contract was executed April 23, 1931. The excavation was completed October 16, 1933, and the concrete lining is now being placed. The contract calls for all the shaft details to be completed in April, 1935, and for the tunnel bore itself to be ready for water November 23, 1934.

Preparing Foundations for the Main Dam and Dike. During all this time, studies were being made to determine the best location and method of construction for the main dam and dike of Quabbin Reservoir. The location chosen for the main dam is just north of the Ware town line and not materially different from that suggested by the Joint Board in 1921. The location for the dike is the same as that originally suggested, slightly south of the divide, between the drainage area of Swift River and Beaver Brook, a tributary of the Ware River. Incidentally the small drainage area of

about 1.4 sq. miles between the divide and the dike is included in the previously stated 186 sq. miles of Swift River drainage area.

At both the dam and dike sites, the sound bed rock is covered to great depths with porous glacial material, and a tight cut-off from the original surface to sound ledge will be required. It appeared that such cut-offs could be constructed most economically by sinking a row of reinforced concrete caissons across the valley and sealing them end to end by a method similar to that recently used, in an even more difficult situation, by the Central Maine Power Company in constructing the Wyman Dam at Bingham, Maine. To demonstrate the feasibility of this method, a contract was let September 16, 1931, for the sinking of two exploratory caissons, one at each site. The success of these exploratory caissons and further experiments in lowering the ground-water level at the dike by pumping, paved the way for a contract which was executed December 28, 1932, for the construction of the complete cut-off at the dike. Under this contract a row of caissons has been sunk across the valley of Beaver Brook, and two additional exploratory caissons are being sunk at the main dam for experimental lowering of the ground-water level.

Incidentally, the ability to complete the entire project in successive steps has made possible a particularly thorough study of various methods of constructing the embankments of the main dam and dike of Quabbin Reservoir. These studies and experiments will result, it is believed, in large savings in construction cost. A very large sum was certainly saved on account of the elimination of much of the contractors' risk and uncertainty in bidding on the caisson core wall for the dike, by the preliminary experiments and demonstration of methods and results to be expected.

In the meantime, in anticipation of the construction of a similar cut-off at the dam, the Swift River was diverted around the site of future construction operations through a tunnel in the hill east of the site. A contract for this tunnel was executed August 1, 1931. Materials excavated from the tunnel and the portal cuts were placed where they will later serve not only as toes of the future dam embankment but as a part of the upstream and downstream cofferdams required when the core wall and embankment are built. The construction was completed and the river diverted through the tunnel June 7, 1933.

In the meantime, studies have been in progress to determine the best method of constructing the dam and dike embankments, and field tests are being made by sluicing materials obtained from suitable borrow areas. A contract was executed February 7, 1934, for the exploration of such borrow areas by steam-shovel cuts.

Relocation of Highways, Transmission Lines and Cemeteries. In the meantime, progress has been made in the construction of highways to replace those the use of which will be discontinued by Quabbin Reservoir. Four contracts for this work have been executed, the first on July 20, 1931. New highways constructed to date have a 24-ft. width of bituminous

macadam surface and 3-ft. gravel shoulders. They are constructed with the coöperation of the State Department of Public Works, which contributes the cost of the surfacing and other incidental items, and upon completion become state highways. To date 5.2 miles of such state highway have been completed south of the reservoir on the route from Ware to Belchertown and 20.8 miles are now under construction west of the reservoir, extending from Belchertown through Pelham, Shutesbury and New Salem.

A contract was also executed June 20, 1933, for the relocation of the 66 000-volt electric transmission line of the New England Power Company, which crosses the reservoir site at three points between Leverett and Ware, requiring 8.5 miles of new line around the southerly end of the reservoir in substitution for 6.6 miles. This contract was only recently completed.

The construction of the reservoir will also drown out 15.8 miles of the present location of the single-track Athol Branch of the Boston & Albany Railroad north of West Ware, including 6 railroad stations, 4 steel bridges and 71 culverts over water courses. There are also 18 public and 26 private grade crossings which will be eliminated.

One important problem is that of removing existing cemeteries in the area to be affected by Quabbin Reservoir. A contract was executed October 29, 1931, for the construction of 4 400 feet of driveways to make a new cemetery, known as Quabbin Park Cemetery, in the Town of Ware about one mile south of the dike site. The development covers 18 acres and includes 1 602 cemetery lots, averaging about 320 sq. ft. or space for about 8 graves in each. A winter receiving-vault was also built to hold 8 caskets. This new cemetery was ready for reinterments August 25, 1932, after which no new burials were allowed to be made in the cemeteries to be moved, of which there are 16 containing 5 365 bodies. The work of removal commenced on December 16, 1927. Generally speaking, bodies are reinterred in whatever cemetery the interested parties request. Several have been moved to cemeteries in other states. It is expected that the great majority will be moved to Quabbin Park Cemetery where lots are assigned in exchange for those in the old cemeteries. To date, there have been removed 1 419 bodies and since August 25, 1932, there have been 38 original interments in Quabbin Park Cemetery. Cemetery removals are being made by a force employed directly by the Commission.

Land Takings. The Commission purchased or took by eminent domain land for all its construction requirements, but no general taking has yet been made in the reservoir area. However, the Commission commenced as early as December 7, 1926, to purchase lands offered for sale in this area. Purchases have been generally confined to the area within one-half mile of the proposed flow line, the entire Prescott "peninsula" between the West and Middle branches of the Swift River, and other areas near the tributary streams. It is estimated that about 100 square miles will be ultimately acquired for Quabbin Reservoir.

Of this total, purchases have been made as follows:

Year Ending Nov. 30.	Land Purchased in Quabbin Reservoir Area — Acres.	Under Option at End of Year — Acres.
1927	10 590	4 560
1928	12 350	3 260
1929	10 880	4 020
1930	5 710	2 340
1931	5 450	1 130
1932	3 695	1 000
1933	5 632	1 731
1934 (5 months to April 21)	2 907	471
Total	57 214	

The Commission has also purchased to date 5 277 acres above the Ware River intake for watershed protection and has 314 acres more under option.

The requirements of land for construction purposes have been as follows: For the pipe lines in the southern Sudbury area the taking was generally an easement 30 to 40 ft. wide across private land. An easement was also taken in public highways. For the tunnel, the taking was generally a strip 50 ft. wide taken in fee. In some cases a 50-ft. easement was taken and included the right to construct and maintain not only the tunnel but also an electric transmission line and a road for access on the surface. In case the tunnel line was located where the transmission line could not follow it on the surface, an additional easement was taken for a detour for the transmission line. At each shaft an area was also taken in fee for the disposal of the rock spoil excavated from the tunnel and for the construction of the contractor's camps. The area taken for these purposes was as follows:

At Shaft.	Area Purchased for Construction Purposes — Acres.
No. 1	On Wachusett Reservoir lands.
No. 2	On Wachusett Reservoir lands.
No. 3	26
No. 4	18
No. 5	22
No. 6	22
No. 7	20
No. 8	On Ware Intake lands.
No. 9	20
No. 10	41
No. 11	28
No. 12	On Quabbin Reservoir lands.
Average	25

For the highway relocations, the taking was uniformly a strip 80 ft. wide taken in fee. Slope easements were taken outside this limit where

necessary. For the transmission line relocation, the easement taken was uniformly 125 ft. wide. For the Rutland-Holden Sewer a permanent easement 15 ft. wide was generally taken, and in addition a temporary easement for construction purposes which increased the total width generally to 40 ft. At the embankment fills, deep cuts and other similar locations, the widths of both the permanent and temporary easements were increased as required. Table 4 gives a summary of lands taken by eminent domain for construction purposes to date.

TABLE 4.— SUMMARY OF LAND TAKINGS.

	Acres taken in Fee.	Acres taken in Easement.
Southern Sudbury Emergency Supply	258.8	29.0
Rutland-Holden Sewer	..	22.9
Quabbin Aqueduct	1115.0	24.0
In Quabbin Reservoir Area:		
Quabbin Park Cemetery	79.8	..
Relocation of Highways	229.7	6.9
Relocation of Transmission Lines	..	12.6
Miscellaneous	295.9	..
Miscellaneous in Ware Watershed	0.3	..
Total	1979.5	95.4

PART III.

DESCRIPTION OF THE PRINCIPAL CONSTRUCTION ITEMS.

Design and Construction of the Quabbin Aqueduct Tunnel.

The construction shafts were generally located on a side hill overlooking an area on which spoil from the tunnel excavations could be dumped and so that the dump cars could be carried along the top of the spoil banks on a slight down-grade and returned empty on the slight upgrade. The contractor's camp buildings as well as his construction camp were also located close to each shaft head. Particular attention was paid to the location of his buildings and the spoil banks at shafts 1, 2, 3 and 4 which were located within the watershed of Wachusett Reservoir. At each of these points the contractor was required to collect the drainage from the entire area in a settling basin and chlorinate the effluent. At Shaft 2, which was close to the Quinapoxet River, a main tributary of the reservoir, the drainage was also required to be filtered through slow sand-filter beds. At Shaft 1, close to the reservoir, plans were made for such filters, but the small amount of tunnel excavation done at this point and the small quantity of drainage involved made it unnecessary to construct them since the bottom of the settling basin acted as a natural filter.

The construction shafts are 14 ft. in diameter and were lined with

concrete as they were sunk to avoid the hazard of falling rock. This size of shaft allowed the installation of double hoists for tunnel excavation with space for wiring, water and compressed-air lines, ventilation pipe and a ladder for emergency exit. This 14-ft. diameter also limited the length of the base line dropped from the surface to determine the direction of the tunnel excavation. At Shaft 3 it was necessary first to sink a concrete caisson through 61 ft. of earth cover to reach sound ledge. At the other shafts ledge was located at depths varying from 3 to 34 ft. and ledge was uncovered by open cut and sheeted excavation of the earth. Each shaft, and in no case less than 135 ft. of the tunnel on either side, generally 200 ft., were excavated by bucket hoist. As soon as the blasting operations were at a safe distance from the foot of the shaft a timber head frame was erected over the shaft and the shaft timbered to support guides for the shaft hoists and other shaft equipment. On the Wachusett-Coldbrook section the skips loaded with tunnel spoil were run onto the hoist platform at the bottom of the shaft, brought to the surface and run in trains to the edge of the spoil bank, dumped and returned by the same route to the tunnel heading. On the Coldbrook-Swift section balanced skips were used in the shafts. The muck was dumped from the tunnel cars into the shaft skips or buckets which dropped into a recess below grade. At the surface, these buckets could be either automatically dumped at the top of the lift into muck cars or crusher bin or kept on an even keel and used to carry men, tools and equipment.

The rock encountered was a compact schist, varying to granite. The author has asked Frank E. Fahlquist, Assistant Engineer in the field organization, to contribute a brief discussion of the geology of the region from his intimate knowledge and experience in the construction. No attempt at any further description of the rock encountered will be made but reference should be had to Mr. Fahlquist's discussion. The rock was uniformly sound except for a few short stretches near the west end. A length of only 1 971 ft. required permanent timber support, which is only 1.52 per cent. of the entire length. In addition, about 450 ft. in scattered locations required temporary timbering. Also in several places, aggregating about 12 050 ft. the seams of the rock tended to disintegrate on exposure to the air so that frequent scaling was necessary to insure the safety of workmen. In such cases the stability of the roof was secured, pending the final concrete lining, by a coating of Gunite usually applied in three courses of about 1-in. total thickness.

At the east or Wachusett end, the borings indicated the necessity of lowering the grade of the tunnel to insure its construction in sound ledge with a sufficient cover to make the roof self-supporting during construction and to insure reasonable water tightness during operation. On account of the west to east slope of the tunnel, drainage during construction was easily taken care of in all the headings that extended westward from the shafts. In each of the easterly headings the grade of the tunnel, except

near the Wachusett end, extends level from the shaft, in order to facilitate the drainage operations and, in general, the grade dips down at the halfway point between shafts. At the Wachusett end, commencing near Shaft 3, to secure the rock cover as previously noted, the grade dips down steeply to a depth of 150 ft. at Shaft 2 and continues on the down grade to a depth of 200 ft. below the normal grade line at Shaft 1.

This tunnel should be classed as a grade tunnel, not a pressure tunnel. Although in operation it will be 100 ft. or more below the hydraulic gradient, nevertheless, the internal water pressure will be less than the external pressure due to the normal ground-water level. This will not be strictly true for a short distance at the Wachusett end where a length of 500 ft. next to Shaft 1 was constructed as a circular pressure tunnel with the lining grouted against the ledge to prevent leakage from the tunnel under the full head of Quabbin Reservoir, which, as previously noted, will be effective through the entire tunnel as far as the control gates at the top of Shaft 1. Another small section 200 ft. long at the extreme westerly end next to Shaft 12 will also be similarly constructed as a circular pressure tunnel in order to prevent excessive leakage from Quabbin Reservoir into the tunnel whenever it may be desired to unwater the tunnel by inserting stop logs at Shaft 12. With these two exceptions, the horseshoe shape is maintained throughout the entire length of the tunnel. The sides are vertical, 11 ft. 0 in. face to face, and the height from invert to arch is 12 ft. 9 in., the cross sectional area being the same as that of the circular sections, 12 ft. 9 in. in diameter.

The tunnel is lined with concrete for smoothness to increase the carrying capacity as well as to prevent disintegration of the rock and to insure a clean unobstructed tube, and excessive external pressures are prevented by frequent vents through the lining. The carrying capa city of the tunnel was found to correspond with a coefficient of Kutter's $n = .012$, and it is expected that this will increase with age to at least .013 but to not more than .014.

Excavation of the Tunnel. The full face method of driving the tunnel was used for most of the excavation at shafts 2 and 4 in the Wachusett-Coldbrook section and at all shafts in the Coldbrook-Swift section. At shafts 5, 6, 7 and 8 in the Wachusett-Coldbrook section, where the heading and bench method was used, the progress was materially slower. In drilling for excavation by the full face method a crew consisted of one foreman, 5 drillers, 5 driller's helpers and 2 steel nippers, using 5 drills mounted on a drill jumbo platform. As the work progressed in the Coldbrook-Swift section the number was reduced from 5 to 4. The position and number of holes varied with the condition of the heading and the kind of rock encountered. Ordinarily about 35 to 40 holes were drilled with delayed timing in the following groups:

	Shafts 2 and 4.		Shafts 9, 10, 11 and 12.	
	Number.	Depth Ft.	Number.	Depth Ft.
"Cut" holes — forming a central cone..	5	10	9	11½ to 12
"Line" holes — defining the break along sides and arch.....	13	8	15	9½ to 10
"Lifter" holes — to throw the muck up and back from the headings.....	4	10	9	9½ to 10
"Reliever" holes — to break up the rock between the "cut" and "line" holes	13	8½	7	9½ to 10

This drilling operation usually required about 3¾ hours. Loading started immediately after the holes were blown clean of dirt and water. At shafts 2 and 4, about 8 sticks of 40- to 60-per cent. dynamite, averaging

FIG. 5.—TUNNEL MUCKING MACHINE IN QUABBIN AQUEDUCT. JANUARY 20, 1930.

about 1.1 lb. per stick, were put into each hole making about 310 lb. per round. At shafts 9, 10, 11 and 12, from 10 to 14 sticks of 40- to 60-per cent. dynamite, averaging about 0.6 lb. per stick, were put into each hole making about 275 lb. per round. Drilling equipment was then taken back to a switch about 1 000 ft. from the heading; lights and air- and water-connections were taken back a safe distance and all the men left the heading which was shot from a switch located about 1 500 to 2 000 ft. from the heading. The net progress varied from about 6 to 8 ft. per round at shafts 2 and 4 and from about 7 to 9 ft. at shafts 9, 10, 11 and 12. The quantity of dynamite used per cubic yard of rock averaged about 6.45 lb. in the Wachusett-Coldbrook section, and about 4.7 lb. in the Coldbrook-Swift section, varying from 3.8 to 5.0 lb., depending on whether the rock was soft or fairly hard. The smaller amount of dynamite used per cubic yard in the westerly section

was due both to the difference in the dynamite and to soft rock encountered, particularly at Shaft 9. Approximately, 2 600 tons of dynamite were used in excavating the entire tunnel.

After the blast, the mucking crew consisted generally of eight men. Air-driven shovel-type mucking machines, capable of working in close quarters, were used at shafts 5, 6, 7 and 8 and electric-driven muckers at shafts 2, 4, 9, 10 and 11. In the east heading from Shaft 12, a scraper mucker was employed. At each shaft about three trains of 4 to 7 muck cars each were required, the number depending on the length of heading. For ventilation, air was exhausted through a 14-in. sheet-metal pipe in the shaft, and along the tunnel within about 200 ft. of the heading generally through spirally wound wood-stave pipe 14 in. in diameter. This wood pipe stood up very well in the face of occasional flying rock at this distance from the heading. Usually the air line to the heading was left open during a shot and until the heading was cleared of enough powder smoke for the mucking crew to start work. This hastened clearing the heading. While drilling, the exhaust from the drills furnished ventilation at the face of the heading.

To keep the drill crew on line, the direction of the tunnel line was transferred from surface monuments by dropping two plumb lines down the shaft. Of course these could not be much over 13 ft. apart. The line was dropped a number of times, when a Sunday or holiday gave opportunity, and projected into the tunnel where it was read on brass vernier scales set into the rock roof. Table 5 shows the accuracy obtained:

TABLE 5.— DIFFERENCES IN LINE AND GRADE AT THE POINTS WHERE THE EXCAVATION MET BETWEEN SHAFTS.

Shaft.	Distance Apart — Ft.	Difference in Feet		Point of Meeting.
		Line.	Grade.	
1– 2	9 902	0.33	0.01	836 ft. from Shaft 1
2– 3	9 167	1.85	0.02	8 537 ft. from Shaft 2
3– 4	8 819	0.81	0.04	1 481 ft. from Shaft 3
4– 5	11 633	0.24	0.02	5 814 ft. from Shaft 4
5– 6	11 102	0.49	0.13	6 329 ft. from Shaft 5
6– 7	9 232	0.31	0.07	4 732 ft. from Shaft 6
7– 8	10 048	0.09	0.14	5 545 ft. from Shaft 7
8– 9	12 412	0.06	0.06	4 998 ft. from Shaft 8
9–10	14 501	0.63	0.11	7 611 ft. from Shaft 9
10–11	15 840	0.12	0.02	8 278 ft. from Shaft 10
11–12	16 757	0.10	0.05	8 254 ft. from Shaft 11
12–Portal	319	0.00	0.02	230 ft. from Shaft 12
Average		0.42	0.06	

The typical contractor's plant at each shaft during the excavation operations consisted of 3 air compressors totalling about 1 800 c.f.m. capacity, one 100-h.p. drum hoist handling the cables through the shaft

head frame, one 60-h.p. blower with a capacity of about 4 000 c.f.m., 2 to 3 mucking machines, pumps, rolling stock, etc.

Progress made in sinking and lining the 14-ft. construction shafts and in excavating the tunnel is shown in the tables 6 and 7, which give the average and maximum progress made and the number of men employed by the contractor in these operations. The shallower construction shafts, 14 ft. in diameter, that is those about 200 to 275 ft. deep, were sunk and lined at the rate of about 25 to 30 ft. per week. At greater depths the progress was slowed up materially so that for shafts over 400 ft. deep the average rate of sinking and lining the entire depth was about 19 ft. per week. The contractor's force for the shaft-sinking operations averaged about 35 to 40 men at each shaft. After the tunnel headings were turned at the shaft bottoms and after head frames and hoisting equipment for the tunnel excavation were installed, a fair average progress of the excavation in each tunnel heading was, in round numbers, 100 ft. per week, although materially less at some of the shafts, as shown in the table. An excellent record was made at Shaft 9 during the week ending October 22, 1932, when 131 ft. were excavated in the east heading and 138 ft. in the west heading. The contractor's force for tunnel-excavating operations averaged about 75 to 100 men at each shaft. This number was later increased by about one-third during the subsequent tunnel-lining operations, working three shifts. The following comparisons are interesting:

In the Scituate tunnel of the Providence Water Supply, most of the excavation was done from a single shaft in 1923 and the first half of 1924. The average progress of excavation from this shaft per heading was about 65 ft. per week and the maximum 82 ft. This tunnel was somewhat smaller than Quabbin Aqueduct but large enough to use the smallest shovel-type of mucker. Practically no support of the rock roof was required.

The Shandaken tunnel of the New York Water Supply was excavated from 7 shafts in 1919–1923. The average progress of excavation per heading was about 65 ft. per week and the maximum about 610 ft. per month. The finished dimensions of this tunnel were 10 ft. 3 in. width by 11 ft. 6 in. height. It was a grade tunnel very similar to Quabbin Aqueduct. Forty-two per cent. of the total length required roof support.

The New York City Tunnel No. 2 was excavated from 16 of its 17 shafts in 1930 and the first half of 1931. The average progress of excavation per heading was about 70 ft. per week. Ordinarily 3 rounds or advances were made in two days in each heading. The maximum progress was made when working a single heading from one shaft. Under these conditions 2 advances per day were made, resulting in a maximum progress of 143 ft. per week. The finished diameter of this tunnel was 17 ft. About 26 per cent. of the total length required roof support.

Lining of the Tunnel. To obtain aggregates for the concrete tunnel lining, rock spoil from the tunnel excavation was crushed and screened at the top of each shaft from which the major lining operations were carried on.

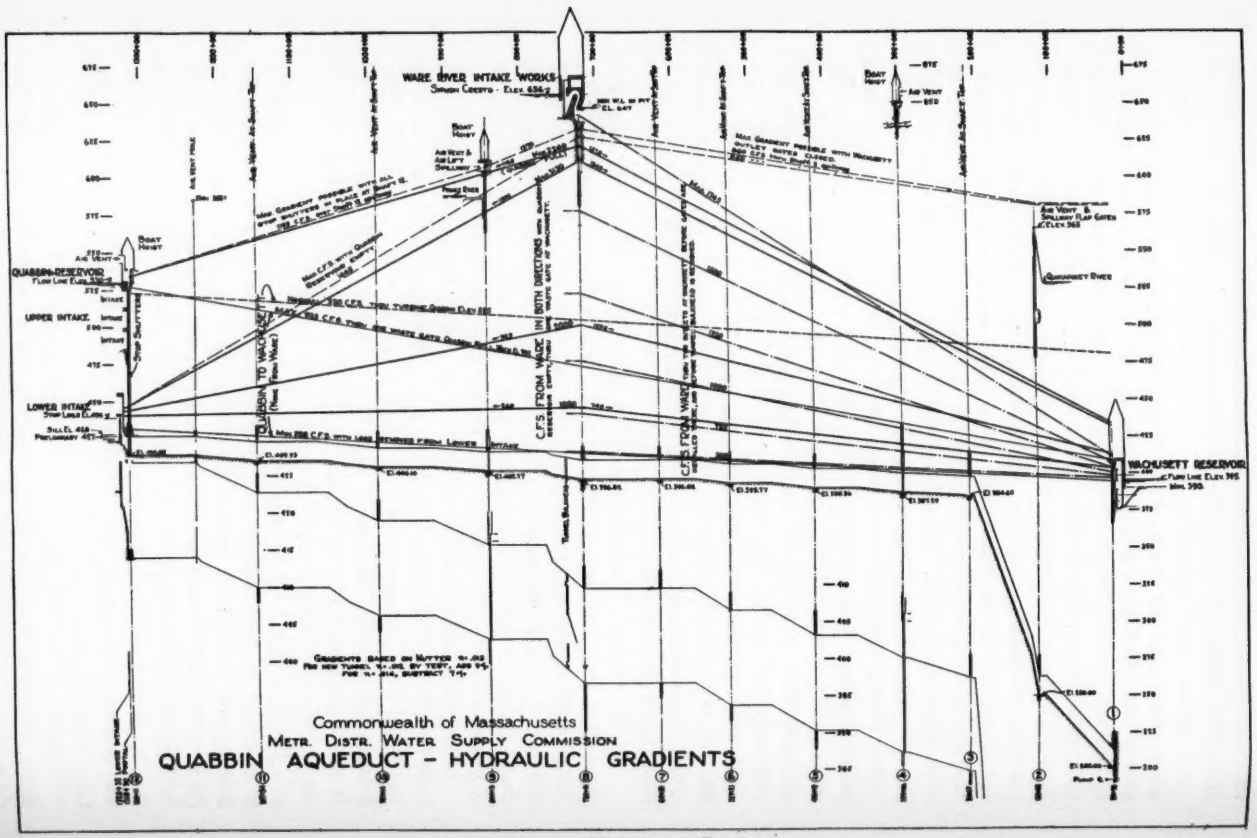

FIG. 6.— HYDRAULIC GRADIENTS.

These were shafts 2, 4, 5, 7, 9, 11 and the portal. Lining is still in progress and practically completed at the latter point, and preparations are now being made for lining from Shaft 10. Also the short section of circular pressure tunnel at the east end of the aqueduct was lined from the mixing plant at the top of Shaft 1. Also concrete in the invert in the west heading from Shaft 8 was mixed at the top of this shaft and dropped in an 8-in. pipe embedded in the masonry of the foundations of the Ware River intake works while these works were under construction by another contractor. At several of the shafts a small shortage of fine aggregate was supplied by bank sand — at Shaft 4 from the tunnel spoil by the use of sand rolls. At Shaft 9 a considerable quantity of sand and gravel was borrowed, and prepared aggregates were hauled from Shaft 10 to supply a shortage of satisfactory tunnel spoil.

The Wachusett-Coldbrook section was lined from three tunnel mixing-and-placing plants, which were moved from point to point along the line of the work. These placed the side walls and arch only, leaving the invert to be placed in a separate operation. The aggregates were mixed at the top of the shaft, dropped to a bin at the bottom and loaded on trains for the mixing-and-placing plant where the water was added during mixing. The 1-cu. yd. mixer discharged into a concrete gun of the same capacity equipped with a piston 32 in. in diameter working through a 66-in. stroke and operated by air pressure of from 75 to 100 lb. per sq. in. This gun forced the concrete into a 6-in. chuting line about 125 ft. long extending over the top of the forms. Each tunnel plant used 200 ft. of collapsible steel forms. These forms were erected on wooden sills set to line and grade several hundred feet in advance of the work. Each 25-ft. section of forms was hinged for collapsing at three points, at the center of the arch and 30 in. from the bottom of each side wall. As each section of the form was collapsed, it was carried on a form jumbo through the interior of the sections remaining in place and again set up at the other end. The concrete was thus poured in one continuous operation, the chuting line discharging above the center of the arch and the wet concrete taking its natural slope down the two sides between the steel forms and the irregular rock excavation.

At the surface plants, the rock from the tunnel spoil banks was handled with a drag line scraper to the crushing and screening equipment, and the coarse and fine aggregates were stored in huge piles above timbered tunnels. In these tunnels the aggregates drawn from the storage piles above were weighed for each batch and on signal from the foot of the shaft, were released onto a belt conveyor, the proper amount of cement added and the dry batch conveyed to the top of the shaft where it was dropped through an 8-in. pipe to a receiving hopper at the foot of the shaft. The invert was placed later in a separate operation after the arch and side wall forms were out of the way. The concrete for this was mixed at the surface involving some rearrangement of plant at the shaft head.

The Coldbrook-Swift section is being lined in a somewhat different

FIG. 6.— HYDRAULIC GRADIENTS.

manner, from four tunnel placing plants. The concrete is mixed at the surface, dropped down the shaft through an 8-in. pipe to a hopper at the bottom and carried to the placing plant in especially designed cars. Lining instead of being a continuous operation is poured in sections requiring end bulkheads. The forms, which are carried ahead on a jumbo in one operation, consist of 100 to 125 ft. of side-wall forms up to about 2 ft. below the springing line of the arch followed by the same length of arch forms, the entire jumbo, including a 60-ft. track incline, being supported on a wide-gage track accurately placed to line and grade. Under this arrangement concrete for the side walls is dumped directly from the cars and that for the arch is placed through a 6-in. chuting line from a concrete gun. To date this same gun has been used to discharge concrete through the arch forms for the placing of the invert which keeps pace with the side walls and arch, but the preparations for lining at Shaft 10 include plans to carry concrete for the invert in cars directly through the arch and side-wall forms in order to speed up the progress.

Progress made in lining the tunnel is shown in Table 8. A fair average progress of the tunnel-lining plant for placing side walls and arch was about 850 ft. per week, for the separate operation of placing the invert as was done in the Wachusett-Coldbrook section about 1 500 ft. per week, and for the complete lining operation as done in the Coldbrook-Swift section about 320 ft. per week in each of two headings worked from one shaft plant.

However, the method of lining used at shafts 9, 10 and 11 had the advantage of completing the entire tunnel section in one operation; also two tunnel plants or sets of forms, one in each heading, were supplied by the mixing plant at each of these shaft heads so that a fair average progress for the shaft concrete plant was 640 ft. per week, twice that obtained by each tunnel plant. Where forms of sufficient length are provided, it is the capacity of the concrete plant at the shaft head that limits the output in each case. The most recent progress, from the portal, was apparently limited to 536 ft. per week solely on account of the 100-ft. length of forms in use at that point. Plant now being installed at Shaft 10 includes two 2-cu. yd. mixers, instead of the 1-cu. yd. mixers previously used, and 125 ft. of forms in each heading.

In comparing the rates of progress made in the Wachusett-Coldbrook and Coldbrook-Swift sections it should be borne in mind that in the former case it was necessary to complete the entire required length of side walls and arch in each heading and then to finally grade and clean up the bottom before the work of placing invert could be started.

For comparison: the Shandaken tunnel of the New York Water Supply, which as previously stated was very similar to Quabbin Aqueduct, was lined from 5 of the 7 shafts. The maximum progress, arch, side walls and invert, was 2 936 ft. per month.

The amount of electric power used in the Quabbin Aqueduct tunnel construction is shown in Table 9 which indicates that throughout the period of excavation and before lining was started, the amount of energy

used for all purposes ran very close to 40 k.w.h. per cubic yard of rock excavated.

Tunnel Measurements for Payment. On all contracts for shafts and tunnel the contractors were paid at unit bid prices computed on the basis of fixed thickness of lining. For each cross-section a so-called "A" line was established as the line beyond which no points of rock were allowed to project. This then defined the minimum thickness of the concrete lining. In sound rock this was fixed at 3 in. on the side walls and invert and 8 in. on the

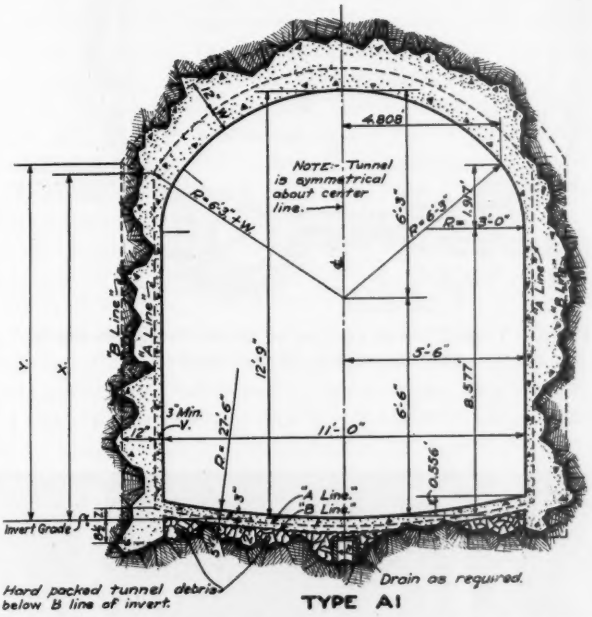

FIG. 7.— TYPICAL CROSS SECTION OF QUABBIN AQUEDUCT TUNNEL.

arch. There was also a "B" line which, except in certain cases where permanent timber support was required, was always a fixed distance from the "A" line, namely 12 in. on the side walls and arch and 2 in. on the invert. The "B" line was generally the pay line for both excavation and concrete, and was intended to represent a fair average of the irregular rock breakage. The actual breakage of rock beyond the "A" line for the side walls and arch averaged 11⅜ in. on the easterly portion of the Wachusett-Coldbrook section, 9¾ in. on the westerly portion of this section and 10⅝ in. on the Coldbrook-Swift section, or a weighted average for the entire tunnel of a little over 10½ in. This means that the actual quantities of rock excavated and of concrete lining placed were respectively 2.6 and 8.4 per cent. less

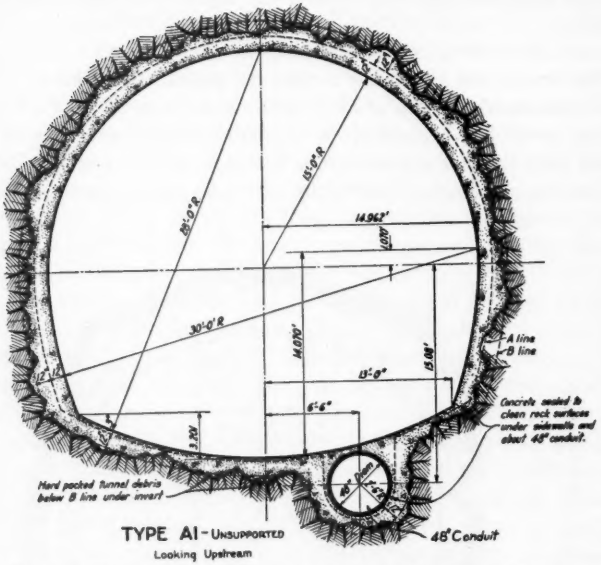

FIG. 8.— TYPICAL CROSS SECTION OF RIVER DIVERSION TUNNEL. Main Dam Site — Quabbin Reservoir.

FIG. 9.—COLLAPSIBLE AND TELESCOPING STEEL FORMS FOR CONCRETE LINING OF SIDE WALLS AND ARCH, QUABBIN AQUEDUCT TUNNEL. NOVEMBER 4, 1930.

than the theoretical or pay quantities reported in the tabular information in Part IV. Except at occasional sections where the concrete invert was carried down to sound rock for a water cut off, the rock débris on the floor of the tunnel was merely graded to a smooth bed, at the required "B" line, on which the concrete was placed. In addition a "C" line so-called was established as the pay line in certain sections where the "A" and "B" lines did not apply on account of the irregular shape of the volumes required to be excavated.

The contractor continually strove to obtain the most economical balance between the various items of the work. If the original drilling and blasting did not break the rock outside the required line and grade, expensive trimming was required at the projecting points. On the other hand if the original breakage was excessive the concrete lining had to be that much thicker. The engineers, following the blasting operations, painted with white paint all points left projecting beyond the "A" line. Enlarging the rock section by additional drilling and blasting was of course expensive but, on the other hand, if too large a section was broken out, it had to be filled with concrete later at the contractor's expense. The distance of 12 in. between the "A" line and the "B" or pay line, for both excavation and lining was determined in advance from experience on other work of this character and is believed to have worked out very satisfactorily in securing fair bidding and adequate compensation to the contractor.

Shaft Head Houses. Upon the completion of the tunnel excavation and lining, the construction shafts were capped in such a way that they can be reopened and used for access at any time in the future in case major construction or repair operations should become necessary. The cover consists of seven removable reinforced concrete slab sections resting on the top of the shaft lining and on two removable beams. This slab is covered by 1½ ft. of earth. An air vent protected by a locked steel grating is built into one side of the shaft at the top and is so designed as to prevent the committing of nuisances and malicious mischief.

A special construction was required at the top of Shaft 2 where four heavy hinged covers were placed in the shaft cap to act as spillway flap gates, air venting being taken care of by four breather pipes. The whole is surrounded by a high steel fence to prevent tampering. The need for a spillway at this point is presented by the very remote but not impossible contingency of the control gates at Shaft 1 being closed at a time when flood flows from the Ware River at Shaft 8 are being discharged into the tunnel at a very high rate. Under such conditions this spillway would reduce the static head tending to lift the top of Shaft 1. At Shaft 2 there is also the possibility of diverting the Quinapoxet River into the tunnel. This river is one of the principal tributaries of Wachusett Reservoir and such a diversion would not increase the drainage area of the supply but would make it possible to store Quinapoxet water in the higher Quabbin Reservoir. This would be particularly desirable if Wachusett Reservoir were well filled at a time

TABLE 6.— QUABBIN AQUEDUCT, PROGRESS IN SINKING AND LINING 14-FOOT CONSTRUCTION SHAFTS.

Shaft No.	1*	2	3	4	5	6	7	8*	9	10	11	12*
Total depth — ft.	221	313	339	457	656	432	411	253	200	408	275	90
Depth of earth cover — ft.	30	25	61	23	19	29	25	3	9	34	24	4
Average weekly progress of excavation — ft.:												
Year ending Nov. 30, 1927	....	21.1	19.2	19.2	19.8	16.5	18.6	....	....	....	....	....
Year ending Nov. 30, 1928	11	....	....	....	....	....	....	13.1	....	....	....	....
Year ending Nov. 30, 1931	....	....	....	....	....	....	....	....	23.5	18.5	33.7	7.3
Maximum depth of rock excavated in one week — ft.	20	41	40	31	47	35	35	25	33	43	44	19
Average number of men employed during typical period, in two shifts:												
1927	....	35	24	37	39	31	35	....	....	....	....	....
1931	....	....	....	....	....	....	....	....	41	37	37	....

* Larger Waterway Shafts Involving Special Construction.

TABLE 7.— QUABBIN AQUEDUCT, TUNNEL EXCAVATION PROGRESS.

From Shaft No.	1	2	3	4	5	6	7	8	9	10	11	12	Portal
Excavated under shaft contracts—ft.													
Year ending Nov. 30, 1927	....	443	.....	.....	.....	.....	251	.....	.....	.....	.....	.....	....
Year ending Nov. 30, 1928	....	1 328	1 196	843	1 367	1 340	1 471	.....	.....	.....	.....	.....	....
Excavated under tunnel contracts—ft.													
Year ending Nov. 30, 1928	....	1 760	.....	888	1 956	2 177	2 038	278*	.....	.....	.....	.....	....
Year ending Nov. 30, 1929	836*	9 614	.....	8 883	8 463	5 988	5 738	7 920	.....	.....	.....	.....	....
Year ending Nov. 30, 1930	....	5 373	.....	2 538	362	.....	547	1 303	.....	.....	.....	.....	....
Year ending Nov. 30, 1931	....	.....	.....	.....	.....	.....	.....	.....	1 149	44*	846†	573†	....
Year ending Nov. 30, 1932	....	.....	.....	.....	.....	.....	.....	.....	12 116	7 563	11 190	4 929	89
Year ending Nov. 30, 1933	....	.....	.....	.....	.....	.....	.....	.....	1 760	7 561	3 780	3 231	....
Total	836	18 518	1 196	13 152	12 148	9 505	10 045	9 501	15 025	15 168	15 816	8 733	89
Average weekly progress of excavation per heading, after head frames were placed — ft.													
Year ending Nov. 30, 1928	....	70	.....	63	80	63	63	.....	.....	.....	.....	.....	....
Year ending Nov. 30, 1929	....	100	.....	85	83	74	58	75	.....	.....	.....	.....	....
Year ending Nov. 30, 1930	....	101	.....	103	78	.....	49	74	.....	.....	.....	.....	....
Year ending Nov. 30, 1931	....	.....	.....	.....	.....	.....	.....	.....	68	.....	.....	.....	....
Year ending Nov. 30, 1932	....	.....	.....	.....	.....	.....	.....	.....	115	92	107	93	....
Year ending Nov. 30, 1933	....	.....	.....	.....	.....	.....	.....	.....	116	93	94	113	....
Maximum progress in one week, two headings in Shaft 2 to 11 inclusive, one heading in Shaft 12 — ft.	....	232	.....	237	192	187	142	.....	269	241	250	136	....
Length requiring permanent timbering — ft.	....	35	324	.....	.....	.....	22	.....	161	1 058	252	119	....
Length requiring Gunite treatment for temporary support	....	.....	.....	2 615	1 825	.....	.....	.....	3 950	3 660	.....	.....	....
Average number of men employed during typical period, in two shifts:													
1929	35	119	.....	125	90	84	67	79	.....	.....	.....	.....	....
1932	....	.....	.....	.....	.....	.....	.....	.....	78	64	74	58	....

* All before head frame was placed.

† Practically all before head frame was placed.

when Quabbin Reservoir was depleted. Such an arrangement would also increase the electric energy capable of being developed at Shaft 1. The existing legislation already provides for such a diversion and a recess has been excavated in the rock opposite a break in the concrete lining of Shaft 2 at a point where such a future connection into the shaft may be made.

Other special constructions were at shafts 4 and 9. At Shaft 4 the water would be backed up from Wachusett Reservoir to a depth varying

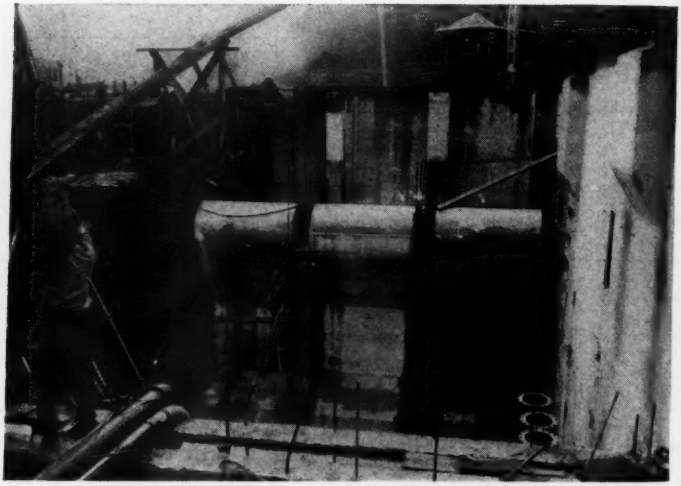

FIG. 10.—CONSTRUCTION OF SIPHON SPILLWAYS IN WARE RIVER INTAKE WORKS.
JULY 2, 1930.

from 2.7 to 7.7 ft. in the 12 ft. 9 in. tunnel, and this is the most easterly shaft which could be reached for inspection purposes by a boat in the tunnel without resorting to pumping. A head house was constructed at the top of this shaft, including provision for attaching a cable hoist to the roof frame so that a boat can be lifted out of the tunnel. This shaft is about 460 ft. deep. No ladder is provided but access is obtained only by bucket hoist. The tunnel at this point is divided by a central pier which supports a platform a few feet above the tunnel arch. This platform provides a landing place for the bucket and also operating space for placing and removing stop logs in the two bays of 4 ft. 9 in. clear span on either side of the central pier. In case it should be necessary for inspection purposes to interrupt the flow in the tunnel temporarily at this point such stop logs would be capable of backing the water up in the tunnel as far as Shaft 10. Permanent hoisting equipment has not yet been installed in the head house. The hoist used to date is mounted on an automobile truck body and is used at other shafts and

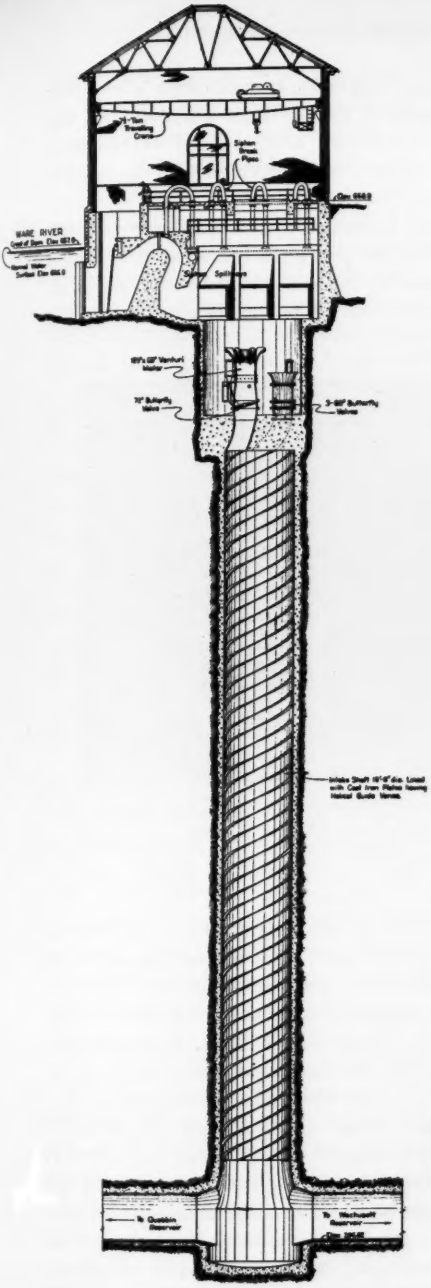
WARE RIVER
Cent of Sec. Elev 1015.
Spillway
Elev. 1048.0
Intake Shaft 12'-6" dia. Lined with Cast Iron Plates Having Velocity Guide Vanes
To Quabbin Reservoir
To Wachusett Reservoir

FIG. 11.

Ware River Intake Works at Shaft 8.

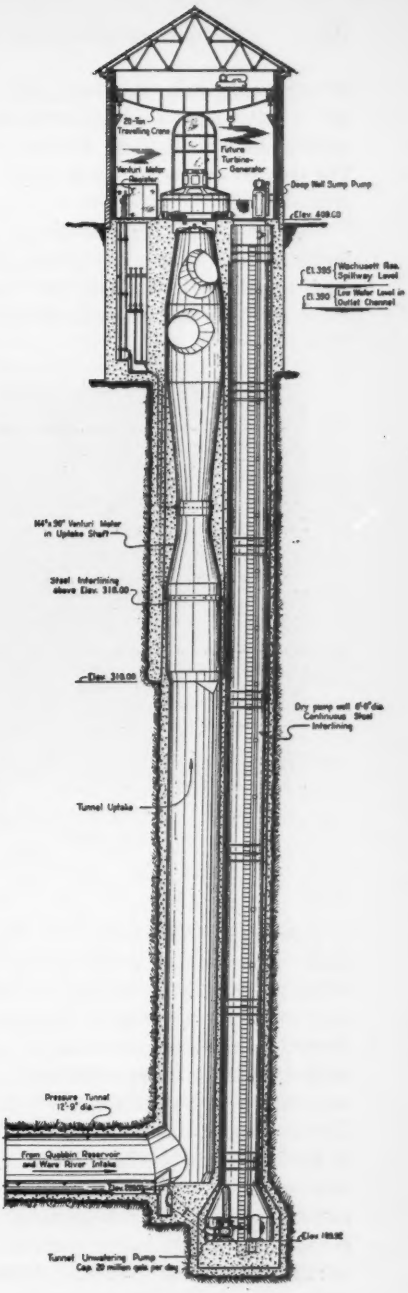
Deep Well Sump Pump
Elev. 409.00
El. 395 Wachusett Dam Spillway Level
El. 390 Low Water Level in Outlet Channel
Dry pump well 6'-0" dia. Continuous Steel Interlining
Tunnel Uptake
Pressure Tunnel 12'-9" dia.
From Quabbin Reservoir and Ware River Intake
Tunnel Unwatering Pump Cap. 20 million gals per day

FIG. 12.

Wachusett Outlet Works at Shaft 1.

QUABBIN AQUEDUCT TUNNEL.

for other purposes. However, a special cable on a special drum is kept for use in the tunnel shafts and is employed for no other purpose, and the automobile hoist is used for this purpose only after special overhauling. The air vent at this shaft is in a false chimney not connected with the interior of the house.

At Shaft 9, a head house will be constructed with similar provisions for bucket and boat hoist, landing platform above the tunnel arch, and stop logs capable of backing up the water in the tunnel above the invert at Shaft 12. A further provision is made at Shaft 9 for the possible installation

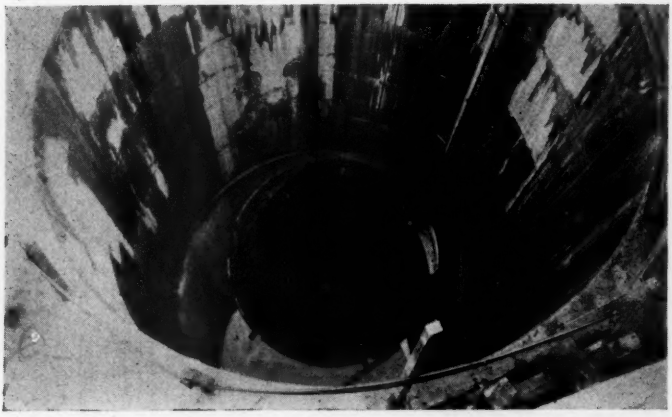

FIG. 13.—CONSTRUCTION OF VALVE PIT AND HELICAL CAST-IRON LINING OF SHAFT 8, WARE RIVER INTAKE WORKS. AUGUST 19, 1930.

of a pump in a chamber in the rock wall at the platform above the tunnel arch which could, if required, supplement the tunnel unwatering pump at Shaft 1. There is no immediate necessity for such pumping plant, and the only provision made in the present construction program is a suction well formed in the concrete lining of the tunnel and a 16-in. discharge pipe embedded in the lining of the shaft. This shaft is not very deep and a 210 ft. steel ladder with platforms every 31.2 ft. will be attached to the shaft walls. The air vent at this shaft will be taken off the side of the top independently of the interior of the head house in the same way as at Shaft 4. However, the top of this shaft is not very much above the hydraulic gradient in the tunnel under certain conditions of operation. The water level will rise to a considerable height in the shaft even when no air is being vented, and the venting of air under such conditions will result in an air lift action which may cause a foamy mixture of air and water to fill the entire top of the shaft. Accordingly, the air vent here is also designed as a spillway with an outlet

to Prince River, an adjacent stream, so that foam and if necessary water can be discharged at this point. In addition to shafts 4 and 9, access to the tunnel by boat will also be provided at the Swift River intake in the head house to be constructed at Shaft 12.

Importance of Air Venting. Although as previously stated Quabbin Aqueduct is not a pressure tunnel in the strict sense of the word, it is precisely that so far as the problem of air accumulations and venting is concerned. Although often neglected, this problem is an important one to consider in the design of any conduit whether a large tunnel or a small pipe. Some interesting phases of the problem as it occurs in a small pipe have already been referred to in the case of the southern Sudbury pipe lines. In large tunnels under high heads, accumulations of air are capable of venting themselves with destructive force. The Ware River intake works are designed to prevent the entrance of air along with the flood flows that are dropped down Shaft 8. On this account, as soon as the water in the tunnel rises sufficiently to seal the bottom of the shaft, the water rises in the shaft about 25 ft. above the hydraulic grade, under a vacuum of about 22 in. of mercury. In spite of this, considerable air does get into the tunnel with the Ware floods and probably more is sucked out of the water by the vacuum effect. This air gradually rises and travels along the top of the tunnel. In order to facilitate travel of this air away from Shaft 8 the tunnel is made level the entire distance to Shaft 7. Any air not vented at Shaft 7 finds vents at five other shafts before reaching the control gates at Shaft 1. Similarly a discharge westward from Shaft 8 finds air vents at each shaft.

The problem is not entirely solved however by finding vents for continuous air flow in the direction of water flow. A much more destructive venting of air can be caused merely by a sudden change in the rate of flow. Mention has already been made of the change of grade or slight hump in the tunnel halfway between two adjoining construction shafts. The distance between shafts is so great that under certain conditions of low velocity an air bubble of large volume could accumulate in the top of the tunnel even on a straight grade and where these humps exist this accumulation may be intensified. This will happen regardless of the elevation of the hydraulic grade, which may be over 150 ft. above the tunnel corresponding to an air pressure of 70 lb. per sq. in. Sudden closure of the gate, for example at Shaft 1, would cause the air which accumulated gradually to be vented suddenly. This could occur at any of the shafts, but most of them are of such height that they each have a very large volume of air cushion to absorb the shock. As an additional safeguard, the access covers in the floors of the head houses at shafts 4 and 9 are hinged at one side and fastened at the other with specially designed clamps, and a fixed louver for ventilation is provided in one window.

In the head house at Shaft 12, however, it would be impractical, on account of provisions to be made for handling stop shutters, screens, etc., and access to the basement, to fasten the floor covers. Hence the tunnel

TABLE 8.— QUABBIN AQUEDUCT, TUNNEL LINING PROGRESS, IN FEET (APR. 21, 1934).

Aggregate from Crushing and Screening Plant at	Side Walls and Arch from Tunnel Mixing and Placing Plants, Aggregates Batched at Shaft Head.		Near Shaft Bottom, Concrete Mixed at Shaft Head.	Invert as a Separate Operation, Concrete Mixed at Shaft Head.		Side Walls, Arch and Invert from Tunnel Placing Plant, Concrete Mixed at Shaft Head.		Average Number of Men Employed at the Shaft Head During Typical Period.	
	Total Concreted in One Stretch.	Average Per Week.		Total Concreted in One Stretch.	Average Per Week.	Total Concreted in One Stretch.	Average Per Week.	Two Tunnel Plants.	One Tunnel Plant.
Shaft 1	.....	....	526	526	....	.....	....	....	....
Shaft 2	9 054	900	241	8 504	1 300	.....	....	193	....
	9 237	862	....	9 365	1 060	.....	....	....	....
	.....	....	....	27	....	.....	....	....	....
Shaft 4	2 831	991	177	5 354	1 780	.....	....	128	....
	3 269	693	....	9 380	1 360	.....	....	....	....
	360	840	....	352	820	.....	....	....	....
	4 752	875	....	207	720	.....	....	....	....
	5 075	772	....	.....	....	.....	....	....	....
Shaft 5	11 079	835	34	11 024	800	.....	....	....	70
	3 977	994	....	5 782	1 850	.....	....	....	....
	.....	....	....	38	....	.....	....	....	....
Shaft 6	.....	....	22	.....	....	.....	....	....	....
Shaft 7	8 307	765	150	8 857	1 940	.....	....	....	96
	4 951	....	....	126	....	.....	....	....	....
	9 952	876	....	10 022	1 280	.....	....	....	....
	845	....	....	302	710	.....	....	....	....
	.....	....	....	22	....	.....	....	....	....
Shaft 8	.....	....	31	4 951	870	.....	....	....	....
	.....	....	....	31	....	.....	....	....	....
Shaft 9	.....	....	....	.....	....	7 419	322	104	....
	.....	....	....	.....	....	6 193	344	....	....
Remaining to be lined.	.....	....	....	.....	....	41	....	....	....
Shaft 10	.....	....	....	.....	....	.....	....	....	....
Remaining to be lined.	.....	....	....	.....	....	16 833	....	....	....
Shaft 11	.....	....	....	.....	....	7 284	270	109	....
	.....	....	....	.....	....	7 941	331	....	....
Remaining to be lined.	.....	....	....	.....	....	27	....	....	....
Shaft 12 and Portal	.....	....	....	.....	....	8 580	536	....	67
Remaining to be lined.	.....	....	....	.....	....	538	....	....	....
Totals	73 689	....	1 181	74 870	....	54 862	....	....	....

TABLE 9.— QUABBIN AQUEDUCT, ELECTRIC POWER USED ANNUALLY BY SHAFT AND TUNNEL CONTRACTORS FOR PUMPING, EXCAVATING AND LINING EQUIPMENT, ETC.

Expressed in Thousand Kilowatt Hours.

The figures for Shafts 1–8, inclusive, include transformer losses. Those for Shafts 9–12, inclusive, exclude transformer losses — of 5 to 6 per cent.

Shaft.	Period of Rock Excavation.	Year Ending November 30							
		1927	1928	1929	1930	1931	1932	1933	1934*
1	Aug. '28–June '29	0	83	320	308	136	.....	.....	.....
2	July '27–June '30	137	848	2,971	3,597	597	.....	.....	.....
3	Aug. '27–May '28	96	546	17	0	0	.....	.....	.....
4	July '27–Apr. '30	185	698	2 957	2 918	616	.....	.....	.....
5	May '27–Jan. '30	273	965	2 298	1 837	231	.....	.....	.....
6	July '27–Sept. '29	102	680	1 641	984	7	.....	.....	.....
7	June '27–Feb. '30	156	878	2 652	2 404	394	.....	.....	.....
8	June '28–Feb. '30	0	247	2 730	3 293	324	.....	.....	.....
9	July '31–Jan. '33	.....	.....	.....	.....	314	2 167	1 330	83
10	Aug. '31–Oct. '33	.....	.....	.....	.....	344	2 719	3 085	10
11	July '31–Apr. '33	.....	.....	.....	.....	307	2 577	3 104	523
12(Portal)	Aug. '31–June '33	.....	.....	.....	.....	322	1 636	1 142	576
Total		949	4 945	15 586	15 341	3 592	9 099	8 661	1 192
Total Rock Excavation — cu. yd.		24 000	120 400	324 700	72 600	25 500	231 800	121 300	None

*4 months to April 1, when 19 042 ft. remained to be lined.

must be vented freely through the interior of the building to fixed window louvers. There will be no high velocity or drop in the level through the Swift River intake works, and in ordinary operation very little air can find its way into the tunnel in the vicinity of Shaft 12. Furthermore to prevent any dangerous accumulations of the air at this point a vent hole at the end of the level stretch east of Shaft 12 has been constructed extending from the tunnel arch to the surface of the ground. This vent was made by drilling an 8-in. hole 152 ft. deep to the tunnel arch and grouting in a 6-in. copper tube with a suitable vent head at the surface.

Temporary Tunnel Bulkhead. Upon completion of the Wachusett-Coldbrook section a massive steel bulkhead was constructed at a point 2 200 ft. west of the Ware River intake works at Shaft 8 capable of withstanding a working pressure of at least 100 lb. per sq. in. The bulkhead was designed to allow the use of the completed easterly section of the tunnel without interruption of the construction work on the tunnel extension to the Swift River. This bulkhead will also allow diversions from the Ware to continue until completion of Quabbin Reservoir to the point where it can store the Ware floods. In order to allow a flow in the eastward direction through the bulkhead under any emergency conditions, particularly to make possible an emergency diversion from a portion of the Swift River watershed in the event of a water shortage, this bulkhead contains two massive swinging doors with specially designed sensitive hinges and is in effect a huge check value.

Tunnel Unwatering Plant — Wachusett Outlet Works. Mention has already been made of the fact that the only tunnel excavation at Shaft 1 was a short length excavated by bucket hoist only, and that no head frame was required at this shaft. In order to provide more time for the construction of the special works required in and at the top of Shaft 1, the contractor pushed the work at Shaft 2 as rapidly as possible and as good progress was made in both headings and no unsound rock or other difficulties were encountered no further excavation from shafts 1 and 3 was required, and no head frames were erected. The west heading from Shaft 2 was holed through to Shaft 3, carried past the bottom of Shaft 3 and holed through to Shaft 4.

The cross section of Shaft 1 was oval in shape, 25 ft. long and 16 ft. wide, inside the first or temporary concrete lining which served for all tunnel excavating and lining requirements. The completed shaft contains the uptake or waterway shaft in the large end of the oval, and in the small end a dry well with a pump chamber at the bottom for use in unwatering the tunnel which, as already noted, was required to be depressed for a length of 3.50 miles at the easterly end in order to ensure a sound rock roof. This dry well was made tight by a continuous steel plate lining. The upper 95 ft. of the waterway shaft also required a steel lining thoroughly embedded in sound ledge for a height of about 65 ft. This was done to prevent the top of the shaft being blown off under the high head that will later be effective due to the elevation of Quabbin Reservoir when control gates are

installed at this point. Near the top of the waterway shaft a Venturi meter was molded in the concrete and mortar lining of this steel plate reinforcement. This Venturi meter is 144 in. by 90 in., and access to the tunnel can be had in an emergency by removing the shaft cap and lowering a bucket through the 90-in. throat.

Above the meter there are two horizontal branches, 7.5 ft. in diameter, on which control gates will be placed, one of these is a waste gate discharging below the level of Wachusett Reservoir, the other is provided as a connection to a penstock for a water wheel which, after the completion of Quabbin Reservoir, can generate power between the Quabbin Reservoir and Wachusett Reservoir levels, a total drop of 135 ft. Much of the total head will be used up in friction in the tunnel, and the maximum rate at which power can be economically generated is about 3 500 to 4 000 k.w. The building has now been left with an open well for such a future installation of power-generating equipment, and no control gates or shaft cap have yet been placed, the two 7.5-ft. openings being of ample capacity to discharge the maximum diversion of flood flows eastward from the Ware River without putting the shaft head under pressure. The Venturi meter also operates for the present with floats which will be replaced by high-pressure mercury chambers when the gates are installed and the full reservoir head is applied to the shaft.

The dry well is 8 ft. in diameter inside a 9-in. concrete lining of the riveted and welded steel shell. This enlarges to a 15-ft. diameter pump chamber at the bottom located 10 ft. below the invert of the tunnel. As the steel plate lining of this shaft was erected from the bottom up there were embedded in the concrete lining two cast-iron pipes, one 20-in. pipe for the pump discharge and one 16-in. pipe for ventilation to the electric motor. The only holes through the steel lining are for the 20-in. pump suction from a well from the bottom of the tunnel, for the 16-in. pump discharge, for the 16-in. motor vent pipe, and for two 1-in. holes for gage-glass connections so that when the tunnel is unwatered the water level can be watched and maintained by throttling the valve on the pump discharge at the bottom of the dry well. This welded steel lining encased in concrete proved to be practically bottle tight, although there is some moisture due to sweating on the concrete lining. A slight leakage drips in through the vent pipe, the bell and spigot joints of which are entirely outside the steel plate lining. A 20-in. guard gate of solid bronze occupies a strategic position between the bottom of the tunnel and the pump well and is secure from any falling objects. This gate is operated from a stand on the main floor by the pull on a continuous chain 418.5 ft. long. To provide for routine or emergency unwatering of this dry well there is installed on the main floor at the top of the well a pump of the deep-well type whose plungers operate in the bottom of a 6-in. steel casing hung in the well. A steel ladder with platforms every 33 ft. provides a more rapid access to the pump chamber at the bottom than the bucket hoist, although the latter is perhaps a more convenient means of exit than this 220-ft. climb.

The blower installed at the shaft head can supply 6 000 c.f.m. of air, which proved to be ample for the 900-h.p. induction motor driving the tunnel unwatering pump. The motor is stored on the main floor at the top of the shaft when not in use. Electricity is supplied at 220 volts through lead-covered cables hung in the well alongside the deep-well pump casing. The pump is capable of delivering 1 400 g.p.m. against a 205-ft. head and is permanently installed at the bottom of the shaft between the 20-in. suction and 16-in. discharge gates. The impeller shaft and the pump-half of the flexible coupling are of stainless steel, and the pump casing is kept filled with oil and inspected periodically. A small drain on the 16-in. discharge line provides a means for draining it in case of clogging of the pump or discharge valve during or after operation, and is ordinarily kept open to prevent any accumulation of head and possible leakage past the discharge valve into the oil-filled pump casing. The level of the oil in the casing is maintained a few inches below an open vent in the top of the casing, and a drain cock in the bottom of the casing maintains the necessary slow drip of water to provide for any very slight gradual leakage past the 20-in. bronze guard-gate on the pump suction.

The outlet to the reservoir is through three 8-ft. bays in which stop shutters can be placed to unwater the tunnel. From the top of the shaft the 20-in. pump discharge-pipe is embedded in the foundation masonry and extends to a point on the reservoir side of the shutter grooves. A travelling bridge crane in the building has a 20-ton hoist for heavy equipment, particularly the expected future generator rotor, and a 5-ton hoist for lighter uses in handling equipment in and out of the deep shaft wells. The travel of the bridge and of the 20-ton and 5-ton hoists on the bridge is effected manually, and the lift of the 5-ton hoist is electrically driven. The main-door frame of bronze is designed so that the side-light sections are removable, if necessary, for taking large pieces of equipment in and out of the building. All the steel roof-truss members are wrapped with wire fabric and coated with Gunite.

The Wachusett-Coldbrook section of the tunnel was unwatered in September, 1931, after the first season of its use for the diversion of Ware floods into Wachusett Reservoir. In spite of the fact that every care was exercised to thoroughly clean the 14-mile length of tunnel draining to the pump, it was considered inadvisable to place a screen over the inlet, during the first pumping operation, due to danger of clogging. A temporary steel rack was erected across the tunnel invert a short distance upstream from the suction well which caught any heavy débris to be removed after the unwatering. Considerable fine débris was removed from the pump casing itself. Several days were spent in warming up the motor and in miscellaneous preliminary work including cleaning out the pump casing. After pumping about 36 hours, the water level was pulled down until the lower end of the tunnel was about half full. The tunnel was then entered by boat lowered through the throat of the Venturi meter and a permanent screen of stainless

steel, a little over 2-in. mesh, was installed over the pump-suction well. In the meantime the tunnel had been inspected by two parties, one entering at Shaft 8 walking eastward and leaving by the bucket hoist at Shaft 4, a distance of 7.95 miles, another party entering at Shaft 4 and following the receding water level eastward and leaving through the Venturi meter at Shaft 1, a distance of 5.29 miles. The entire work was found in excellent condition. To facilitate inspection work, station locations in the tunnel are marked every 500 ft. and at shaft bottoms by 9-in. by 4-in. glazed tiles bolted to the walls.

Design and Construction of the Ware River Intake Works. At Shaft 8, for tunnel construction purposes, the first or preliminary lining was 20 ft. in diameter, and the tunnel contract included enough of the concrete building foundations to act as a cofferdam to prevent high water in the river from getting into the shaft. The tunnel contractor also placed the final shaft lining, which consisted of cast-iron segments 9 ft. 9 in. by 3 ft. 8 in. by ⅞ in. with helical ribs which formed a sort of continuous 6-pitch screw thread projecting 6 to 10 in. into the waterway. There was no machine work on these castings, but they fitted together so neatly by means of lugs and sockets that they served as forms to hold the concrete backing without any calking of seams. This cast-iron lining was designed for the purpose of preventing destructive scour at the sides and bottom of the shaft due to the tremendous energy in this fall which is 50 per cent. higher than Niagara Falls. The valves at the top of the shaft discharge tangentially at a velocity of 30 to 40 ft. per sec. against the shaft walls, and without any great increase in velocity this lining maintains the circular motion to the bottom of the shaft so that the water is at all times pressed firmly against the walls by centrifugal force and has no opportunity to impinge directly against any surface. Before the final designs were made, this action was studied with the help of tests on a small model in the hydraulic laboratory of the Worcester Polytechnic Institute.

Among other pipes embedded in the concrete masonry back of the shaft lining, small pipes were installed to and from an air-lift fitting at a flowing seam encountered in the ledge during excavation, and by this means an excellent water supply for use in the intake building was obtained.

A separate contract was let for the diversion dam and the substructure of the building. The dam across the river is curved in arch form in order to increase the spillway length and also because this particular design enhanced the beauty of the entire structure. The spillway is 208 ft. long at elevation 657 above Boston City Base, 127 ft. above the flow line of Quabbin Reservoir and 262 ft. above that of Wachusett Reservoir. The diversion dam is about 17 ft. high, and this head is used solely for the operation of weirs and meter which discharge down the stream the required natural flow up to 85 m.g.d., and for emergency by-pass gates of greater capacity.

The diversion of flood flows above this specified limit is made through

a battery of 9 siphon spillways varying in capacity from 60 to 550 c.f.s. under a 9-ft. head. These siphons skim off the flood flows above the 85-m.g.d. rate and hold the level of the small diversion reservoir practically constant regardless of the rate of diversion. These siphons discharge into a central valve pit, the water level in which varies with the head required on the siphons. The water in the pit acts as a seal to prevent the entrance of air into the tunnel. In the bottom of the pit are four butterfly valves of the Dow-disc type which control the discharge down the shaft. They are operated by oil pressure of about 50 to 75 lb. per sq. in. These valves are opened successively one after another as their respective capacities are required under an increasing rate of discharge. Ordinarily only one, 6 ft. in diameter, is needed since this can take care of flows up to about 950 c.f.s. The other three, each 5 ft. in diameter, are available for the higher flows, although all four will not be used except when the available discharge is sufficient to tax the capacity of the tunnel in both directions. The maximum capacity after Quabbin Reservoir is completed and gates are installed at Shaft 1 will be about 1 000 m.g.d. eastward to Wachusett Reservoir and at the same time about 750 to 1 030 m.g.d. westward to Quabbin Reservoir depending on whether that reservoir is full or empty. To date, the maximum quantity discharged in 24 hours eastward, to Wachusett Reservoir only, is 682 m.g., the maximum discharge for a short period being at the rate of 1 200 m.g.d., which was sent through the tunnel on April 1, 1932, for test purposes.

A travelling bridge crane in the building has an electrically driven hoist which, when the valve pit is unwatered, can be used to lower men and materials through the shaft into the tunnel. This hoist can be rereaved in a ratio of one to four for use at a 7½-ton capacity for handling the valves and other heavy equipment. The travel of the bridge and of the hoist along the bridge is effected manually. The side-lights of the bronze main-entrance door can be removed to pass large equipment as in the case of the Wachusett Outlet building. Here also the roof-truss members were covered with a protective coating of Gunite.

The design of many portions of the entire project and particularly of these intake works involves a number of interesting features which cannot very well be discussed in a general description of this sort. Some of the more interesting details include a vacuum system piped throughout the building, connected to the Venturi meter which measures the 85-m.g.d. discharge down the river, at a point just downstream from the meter throat. This vacuum provides among other things a means for rapid priming of the siphons. The entire control mechanism for the four main valves in the bottom of the pit is on a single small panel which contains for each of the four units a small oil sleeve-valve and a simple device for preventing hunting or racing so that the valves adjust their openings easily and smoothly to the fluctuations of the flow in the river. Another small mechanical brain, in the form of a floating dash-pot, practically anticipates

the small rise and fall of the river and adjusts the speed of opening and closing the valves accordingly. By the use of specially designed super-sensitive float-switches operating on a differential head of a small fraction of an inch, the water level behind the diversion dam is kept practically constant during the entire range of operations from a diversion of nothing up to the entire capacity of the works. These switches assist the rapid priming of the siphons one after another as needed. The operation is entirely automatic with an operator on hand only during the ordinary working hours of the day.

FIG. 14.—CONSTRUCTION OF LOWER SWIFT RIVER INTAKE TO QUABBIN AQUEDUCT TUNNEL. OCTOBER 17, 1932.

When flood flows have been available, the diversion has been continued without attendance nights, Sundays and holidays.

Design of the Swift River Intake. The intake works at Shaft 12 will consist of an upper intake to be constructed above the shaft itself drawing from the upper water levels at depths from 5 to 45 ft. below the reservoir flow-line and a lower intake now under construction extending 200 ft. from the west tunnel portal, and 520 ft. from the shaft, drawing normally at depths from 76 to 91 ft. or in an extreme drought to 102 ft. below the reservoir flow line. A coarse rack of reinforced concrete bars with 12-in. open spaces is built over the entrance to the lower intake. After the reservoir starts to fill, the only access to this rack will be by a diver, who may land and work on its flat top. The draft from both intakes will be controlled by shutters operated from a head house over Shaft 12. No gates, Venturi meters or other works will be installed. They would in any event be im-

practical on account of the two directional flows through the tunnel. The flow from the Ware River intake works westward into Quabbin Reservoir can be as much as 1 030 m.g.d., that eastward from Quabbin Reservoir to Wachusett Reservoir can be as much as 600 m.g.d. Emergency spillway wells from the basement of the building to the lower intake are provided, to by-pass any stop shutters that can be placed, so that in no event can a flood discharge from the Ware River exert a lifting pressure at the top of Shaft 12.

Design and Construction of Stream Control Tunnel at Main Dam Site. The only important item of construction that has been accomplished at the

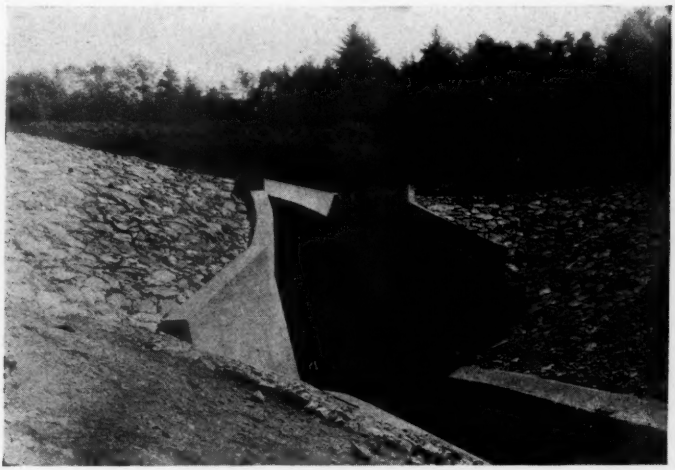

FIG. 15.—COMPLETING PAVING AND RIPRAP AT ENTRANCE TO RIVER, DIVERSION TUNNEL, MAIN DAM SITE, QUABBIN RESERVOIR. JUNE 14, 1933.

main dam site is a tunnel through the rock under the west abutment of the dam, through which the river has been diverted from the site of future construction operations. The Swift River valley has not experienced any floods of great magnitude during the short period of record covering the last twenty years. However, the possibility of the occurrence of such floods is recognized, and the stream control tunnel, 30 ft. wide by 28 ft. high, 1 230 ft. long is required to make sure that the construction of the main dam can be completed expeditiously and all risk of damage from flood flows eliminated. The sound ledge on the side of the valley at this site is covered with huge irregular masses of glacial drift which made it necessary to excavate about 289 000 cu. yd. in the approach and discharge cuts. Furthermore, it was impracticable to secure a straight alignment for the tunnel. Under the

circumstances, the most efficient hydraulic properties for the discharge of the necessary 20 000 to 25 000 c.f.s. at high velocity were secured by building the tunnel in a straight line to a point near the downstream portal. Here a smooth gradual Venturi-like enlargement was obtained by lowering the floor of the tunnel about 6 ft. and raising the arch about 4½ ft., keeping the same dimensions of the arch for simplicity in form work. Under flood-flow conditions taxing the capacity of the tunnel, this enlargement will reduce the velocity without material loss of head. The alignment then contains a 90-deg. bend near the downstream end, but the losses in this bend are materially reduced on account of the decrease in velocity in the enlarged

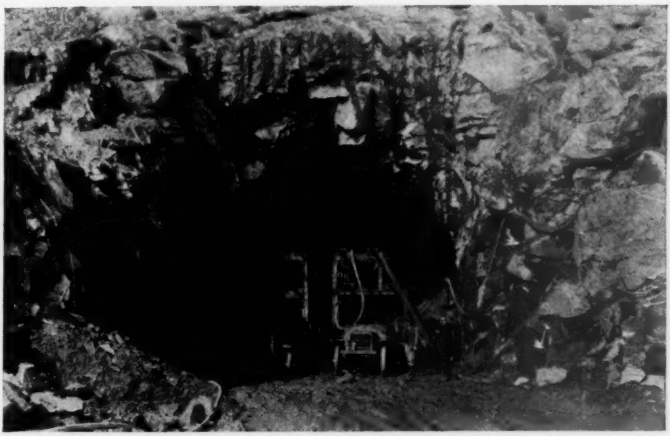

FIG. 16.—DRILL CARRIAGES AT DOWNSTREAM PORTAL OF RIVER DIVERSION TUNNEL, MAIN DAM SITE, QUABBIN RESERVOIR. MAY 2, 1932.

section with depressed invert which is carried completely around the bend to the downstream portal.

After the main-dam embankment is built it will be necessary to plug this diversion tunnel with a solid block of concrete 40 ft. in length. In anticipation thereof 2 piers, at a point about 375 ft. from the upstream entrance, cut the waterway into 3 bays of 9-ft. span for the insertion of stop logs. To minimize the loss of head past these piers, the tunnel section upstream from the piers is 30 ft. high or 2 ft. more than downstream and the net water area past the piers is the same as that of the main tunnel downstream. Like the Ware River intake already described, many interesting features of the design cannot be discussed in a general description such as this. Provision had to be made not only for the future plugging of the conduit but for the needs of future operation and the release of stored water to meet the requirements of Massachusetts Legislation and of the War Depart-

ment previously referred to. These provisions include a vertical shaft, above the piers and the future plug, which will serve as a high-level intake, also a 48-in. pipe which had to be embedded at this time in the concrete side walls and invert of the tunnel extending from this future intake through to the downstream portal, where foundations were prepared for future control works. One of the novel features of the design is provision for maintaining the flow during future plugging operations by siphoning the river through a 48-in. pipe in order to by-pass the construction work required to complete the 40-ft. plug.

All materials excavated from the approach and discharge cuts were used to construct cofferdams, both upstream and downstream, forming the toes of the future dam. These were placed entirely west of the railroad which runs along the east bank of the river. They will later need to be extended across the entire valley, when the construction of the dam foundations is undertaken. The rock excavated from the tunnel was used to construct portions of the downstream toe and for riprap so far as needed, the balance being stored for future similar use. In excavating the tunnel, a top heading from about 2 ft. below the springing line of the arch to the roof was first excavated about 840 ft. from the downstream portal face followed by a pilot cut about 6 ft. square which was holed through to the upstream portal for ventilation only, the entire enlargement to full section being excavated from the downstream portal. The rock was sound throughout the entire length of the excavation, and the roof required no timber or steel support or Gunite protective treatment. The size of the tunnel allowed mucking operations with a steam shovel and heavy trucks. Special drill carriages were erected on truck bodies and thus moved rapidly and conveniently to and away from the face. The heading excavation was started January 4, 1932, holed through in 13 weeks, 955 ft., and the entire excavation of the full section was practically completed 11 weeks later, the shaft being worked from the tunnel up. The lining was cast with steel forms and required 12 weeks. A cut and cover section 275 ft. long extends to the upstream face of the embankment of the dam, the total length of the conduit being 1 230 ft. The tunnel was completed on February 8, 1933. Complete diversion of the river from its old bed through the tunnel was effected June 7 by small earth dams across the river bed near the end of the approach and outlet channels.

Design and Construction of Caisson Core Walls. The water-tight cutoff required along the center line of the dike from the original surface through the porous, glacial material to sound ledge is 130 ft. deep below the brook level. A similar cutoff at the main dam will need to extend to a depth of at least 110 ft. below the river bed. An exploratory caisson was first sunk, in the less difficult of the two situations, at the dam, a location being selected about 450 ft. from the river and at a point where ledge was encountered about 70 ft. below the river level and 102 ft. below the original surface, but about 45 ft. above the lowest point of the rock valley. A second exploratory

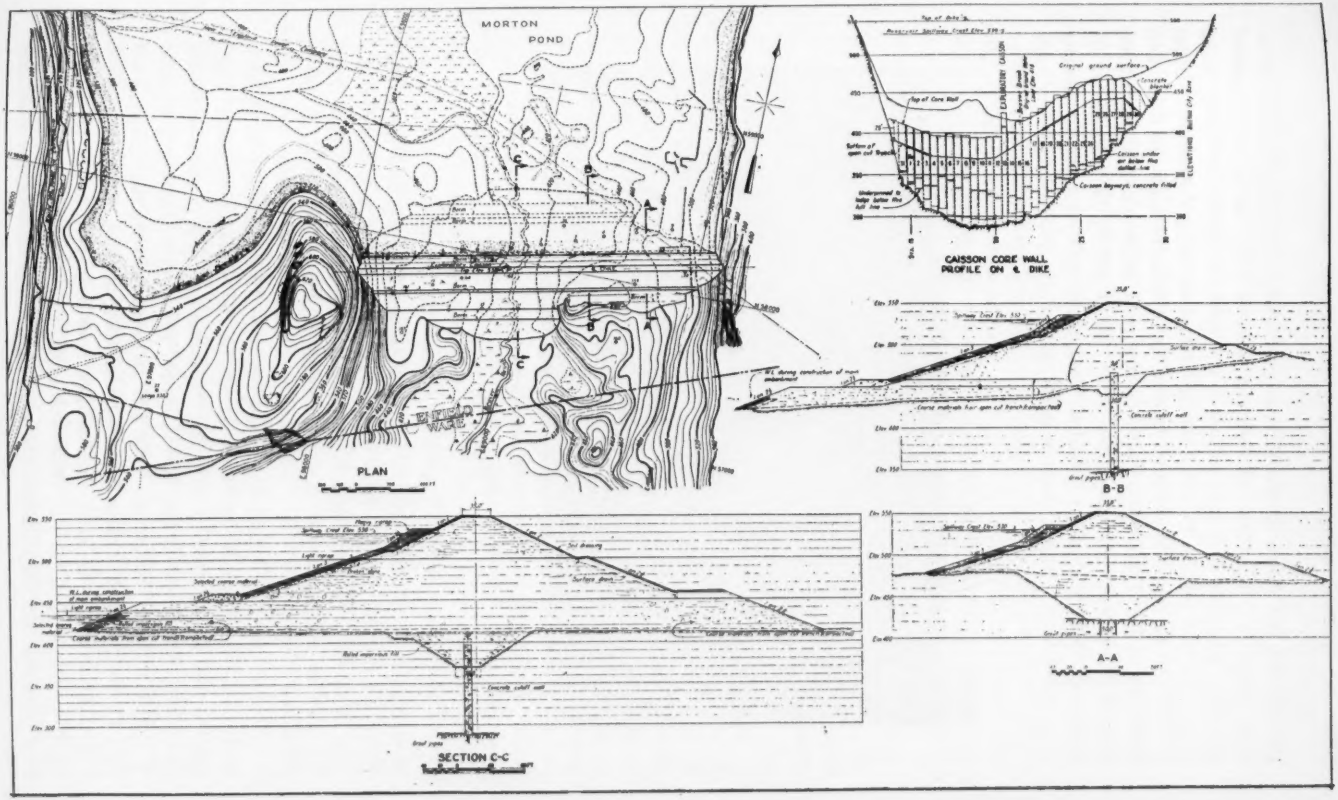

FIG. 17.— QUABBIN RESERVOIR — DIKE.

caisson was put down at the dike at practically the deepest point in the rock gorge 127 ft. below the original ground-water level. This required alternate pumping and excavation of the caisson and an auxiliary pump sump near the caisson to lower the ground-water level in successive stages so that the air pressure required to seal the caisson to ledge did not exceed 48 lb. per sq. in. The caisson itself was first sunk as deep as possible without resorting to excessive air pressures, about 30 lb. per sq. in. being used. By pumping from the working chamber, the ground-water level in the vicinity was then lowered, and a sheeted pump sump about 26 ft. square was ex-

FIG. 18.—CONSTRUCTION OF CAISSON CORE WALL TO SOUND LEDGE ON CENTER LINE OF QUABBIN RESERVOIR DIKE. JUNE 15, 1933.

cavated at a point about 100 ft. from the caisson to a depth of about 50 ft. below the original ground-water level. The pumps were then transferred to this sump and the lowering of the ground-water level continued during the further sinking of the caisson to ledge, requiring 48 lb. per sq. in. air pressure.

These first two exploratory caissons were 32 ft. long by 12 ft. wide, with two 7-ft. diameter wells for access to the working chamber and a central well to be used in pumping experiments. The results of sinking these caissons indicated that a 9-ft. width would give a working chamber with ample room, that access wells 4 ft. in diameter were large enough, and that a greater length of caisson could be handled without difficulty. Accordingly, the caissons designed for the dike core-wall were made 45 ft. long by 9 ft. wide with three 4-ft. diameter wells. In sinking these two exploratory caissons, various experiments were made to determine the feasibility

of combining pumping operations for lowering ground-water level with the sinking operations. It was found impracticable to sink caissons through this glacial material, containing many boulders of all sizes, without forcing the water level down so that considerable quantities of air escaped below the cutting edge of the shoe. It was also found that the action of the air escaping through the ground often had a tendency to separate the fine materials which were carried along in any underground water courses. These fines had a tendency to become tightly packed and clog any suction intakes above the cutting edge from which an attempt was made to pump the ground water. It was demonstrated that no pumping could be done from a caisson while it was being sunk under compressed air, but that all pumping for the purpose of lowering the ground-water level should be done from such an intake placed at an elevation lower than the cutting edge of any caisson being sunk in the immediate vicinity.

Accordingly, when the exploratory caisson at the dike was sealed to the ledge, four pump-suction intakes were permanently embedded in the seal consisting of pipes projecting into areas filled with selected coarse material. One 6-in. pipe intake was constructed under the cutting edge on each side and one 8-in. on each end. These were carried up into the bottom of one of the 7-ft. diameter wells. These pump-suction pipes were independent of each other so that upon completion each could be grouted to prevent any possible open waterway through the core wall. A pump was installed in the well and commencing the first week in September, 1932, water was pumped from the ground at the rate of 1 250 g.p.m., in addition to about 500 g.p.m. pumped from the open sump nearby, until this operation was taken over by the contractor for the caisson core wall on January 21, 1933. Meanwhile the brook was carried past the site in a small wooden flume in order to reduce seepage from the brook bed into the ground. When the contractor took over the work the ground-water level in the vicinity had been lowered 41 ft., and considerable data had been made available to prospective contractors as to the possibility of further unwatering by the installation of more pumping capacity.

A scheme for ground-water pumping operations was included in the contract plans by which the contractor could be paid for sinking at selected locations two of the caissons from the original surface, before excavating any open trench across the valley, in order to start working pumps in these caissons as soon as possible. As soon as the contract was let, the contractor took over the pumping operations and installed pumps of greater capacity in the deep dry well which had been prepared for him in the exploratory caisson. Instead of sinking the two caissons as proposed, the contractor elected to pump intermittently at shallower depths from a number of caissons at different stages of the sinking, starting first on the east side. After sinking one of these caissons by open digging until the ground-water level was reached and to such further depth as was convenient without using excessive air pressure, the sinking of this caisson was stopped for a time; a

specially designed steel diaphragm was installed in the bottom of one the circular access wells to the working chamber and a temporary pumping plant was installed in this well, pumping directly from the working chamber with an ideal large gravel floor surface for a suction screen. By this method the work progressed very satisfactorily. In addition to pumps installed above the temporary steel diaphragms in caissons not yet sunk to ledge, permanent suction intakes embedded in the concrete seal between the cutting

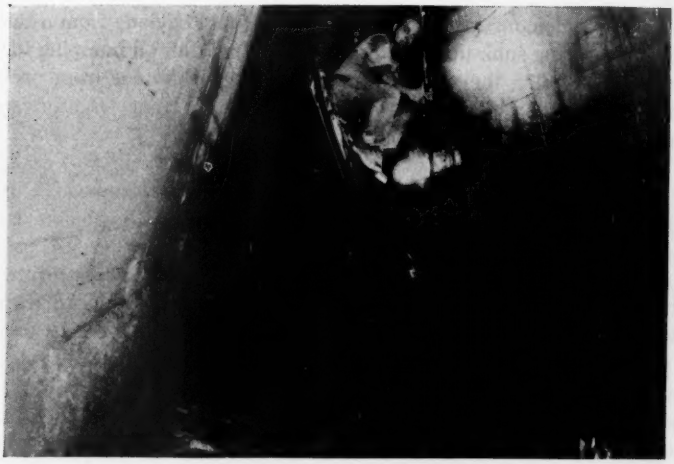

FIG. 19.—WORKING CHAMBER OF DIKE CORE WALL CAISSON AFTER COMPLETION OF SINKING AND UNDERPINNING FROM CUTTING EDGE TO LEDGE.
AUGUST 4, 1933.

edge and sound ledge were installed in three completed caissons similar to those installed in the first exploratory caisson.

The total rate from all of these pumps has averaged about 3 200 g.p.m. for the last year. The ground-water level in the deep portion of the valley in the exploratory caisson has been lowered 96 ft. from the original ground-water level, a gain of about 55 ft. since the pumping was taken over by the contractor. The valley is about 1 500 ft. wide, and the natural slope of the original bed of the brook, which flows out of Morton Pond, about 1 200 ft. upstream from it, is about 1.5 ft. per thousand. The ground-water level has been lowered 56 ft., 220 ft. upstream, 37 ft., 800 ft. upstream, and 16 ft., 2 350 ft. upstream, and Morton Pond itself has been lowered by the underdraining operations about 9 ft. The ground-water level has been lowered 35 ft., 1 050 ft. downstream, and 14 ft., 2 050 ft. downstream. Our computations indicate that if the contractor had used higher air pressures on two caissons sunk at the start from the original surface the entire distance to

sound ledge in order to establish pumps at fixed strategic points, he might have accelerated the lowering of the ground-water level and effected a slight economy in his operations. Table 10 recording sinking progress shows that the air locks were put on quite a bit sooner than the pressures finally required to seal to ledge would indicate as necessary if the pumping had been started earlier.

It was estimated that an economical balance between the cost of the caissons sunk from the bottom of an open trench across the valley and the cost of excavating such a trench and refilling it finally with suitable impervious material would be obtained at a depth of about 45 ft. Accordingly, this depth to the bottom of the open-cut trench was specified in the contract as the pay line from which caissons should be sunk to sound ledge. The excavation of this trench was kept well in advance of the placing of the caisson shoes. Suitable previous materials from the excavations were used to construct embankments across the valley both upstream and downstream which will serve as toes for the future dike embankment. A layer of impervious material on the upstream toe converted this into a cofferdam which will cut off the entire flow of the brook when the construction of the future dike embankment is begun. At present an 18-in. pipe serves as a by-pass through the operations, but the only flow in the brook at present is that discharged from the pumps for ground-water lowering. A considerable quantity of material, too fine for use, was wasted in spoil areas downstream from the site.

The steel shoe or cutting edge for the first caisson was set March 22, 1933. This caisson was sunk from an elevation just above the original brook level to a depth of 48 ft., before ground water necessitated the installation of air locks. The contractor then continued with the caissons in the east half of the valley, sinking first every alternate caisson and later the intermediate ones. The concrete was poured in steel forms erected in 16-ft. lifts using two 8-ft. panels each, and one lift of about 210 cu. yd. was poured daily. Aggregates for the concrete were obtained from an excellent gravel deposit about one mile from the site. A central crushing, screening, washing and batching plant was installed just downstream from the site to which cement was hauled in bulk from the railroad siding at West Ware, three miles distant. Mixing was done enroute to the caissons in two 5-cu. yd. transit mixers. As soon as each caisson was sunk to the point where its cutting edge touched ledge, concrete underpinning walls were built extending until the entire area of the ledge was exposed and clean. A concrete seal was then poured to a height of 3.5 ft. above the cutting edge, leaving a sufficient area in the working chamber to operate drilling and grouting equipment by which seams in the ledge, pump suction openings, etc., were grouted. All the working chambers will ultimately be completely filled with concrete.

In the easterly half of the wall the underpinning and sealing to rock were usually started at the uphill end of the caisson and extended a little more than half the horizontal length or so as to furnish a firm support for the

weight of the caisson, and the seal was not completed until the next adjoining caisson on the lower or downward side had been similarly sealed on ledge and a firm foundation. Such completion of the seal was thus carried through to the adjoining caisson forming a continuous cut-off across the valley at the ledge. Above the ledge the cut-off was made continuous by filling the end joints between adjoining caissons. As previously stated, the caissons were 45 ft. long and 9 ft. wide. They were placed 46.5 ft. on centers, leaving an end clearance of 18 in. The ends were squared for a height of 7 ft. above the cutting edge. Above this an 18-in. recess was cast in each end. Hence the working space available for excavating and filling the end joints averaged 4½ ft. in width. This proved ample in spite of the fact that it was impossible to sink the caissons precisely on the vertical alignment and some of the end joints were narrower and some wider. The end joint-space would also be distorted in a case where one of two adjoining caissons leaned sideways from the true alignment. Such distortion in no case amounted to more than a few feet, the maximum about 5 or 6 ft. at the top and 3 or 4 ft. at the bottom, and no difficulty was experienced in excavating the joints and supporting the outside earth with timbering. When the joint excavation reached the ground-water level the excavated portion was filled with a concrete cap, keyed to the caissons as necessary to prevent blowing off, leaving a 3-ft. circular well at the top of which an air lock specially designed for the close quarters was installed to permit continuing the excavation to the ledge, or to concrete previously placed as described before.

The wall across the valley requires 30 of these caissons in addition to the first exploratory caisson. All of these have been sunk and sealed to the ledge, the first shoe having been set in March, five more in April, five in May, four in June, nine in July, two in August, three in September, and the last one in October, 1933. The only other work of caisson sinking remaining under this contract is two additional exploratory caissons at the main dam site which are now being started in the deepest part of the gorge where experimental work in the lowering of the ground-water level will be continued.

Each caisson in the dike core-wall was required to be left projecting at least 25 ft. above the bottom of the open-cut trench. Where the caissons were seated with the tops too low, they are being extended by pouring additional concrete to the required height. The end joints have been completed, April 21, 1934, to an aggregate height of about 80 per cent. of the required total. At the extreme ends of the row of caissons, a solid concrete wall of the same width is built in sheeted trench or open cut to ledge and is extended until it meets the sloping ledge hillside, and a concrete blanket covering the ledge and sealing all seams for a width of 20 ft. will later be extended up the hill along the center-line of the impervious core of the future dike embankment. Grouting seams in the ledge through drill holes generally about 8 ft. apart and 15 ft. deep is now under way.

Table 10 giving the depths to which the caissons were sunk shows the

level reached by each before and after ground water was encountered, necessitating the installation of air locks. This does not indicate the full extent to which the work benefited by the lowering of the ground-water level, since this lowering continued to reduce the necessary air pressures after the locks were installed. During the sinking operations, the average rate of sinking in the open was 8.63 ft. in 24 hours and under air 2.51 ft. up to November 30, 1933. Although the maximum depth of sinking below the original ground-water level was 130 ft., the pressure (except for the exploratory caisson which reached 48 lb. per sq. in., as previously described) has rarely exceeded the 18 lb. per sq. in. under which men are permitted to work 8 hours a day, as many as in the open air. The maximum pressure reached on the caisson core wall contract was 25 lb. per sq. in.

In no case was it necessary to add pig iron or other excess weight to sink the caissons. However, whenever boulders too large to be handled in the bucket hoist or through the muck lock were encountered they were broken up with dynamite, and this blasting shook up the ground and helped settle the caisson. In some cases also, dynamite was exploded in the ground outside the cutting edge for the sole purpose of shaking up the ground and assisting in the sinking operation. In this connection it should be noted that the use of too much dynamite, particularly before the cutting edge approaches the required depth, may result in settling and packing the ground against the caisson so firmly as to be of doubtful benefit in completing the sinking. Several caissons were stopped a few feet before the cutting edge struck ledge because the Contractor chose to excavate the remaining distance behind underpinning walls rather than to carry the sinking further. The weight of the 25-ft. height of caisson required to be left projecting above the bottom of the open trench was in all cases available, even in the early stages of sinking, to such extent as the contractor chose to add this weight.

The following tabulation shows the distribution of the total Electrical Energy used on the Dike Caisson Core Wall including sinking of Exploratory Caisson and all pumping:

		k.w.h.			k.w.h.
1932	Feb. Exploratory Caisson	6 200	1933	Mar. Caisson Core Wall	68 130
	Mar.	53 700		Apr.	98 100
	Apr.	47 500		May	146 800
	May	67 900		June	158 800
	June	75 500		July	167 600
	July	143 700		Aug.	170 200
	Aug.	27 600		Sept.	216 900
	Sept. Interim Pumping	29 300		Oct.	214 300
	Oct.	43 000		Nov.	246 200
	Nov.	41 300		Dec.	254 000
	Dec.	42 300			
			1934	Jan.	262 400
1933	Jan. Caisson Core Wall	38 200		Feb.	247 400
	Feb.	42 100		Mar.	258 600

TABLE 10.— QUABBIN RESERVOIR DIKE, PROGRESS IN SINKING CAISSONS.

Caisson Number.	Date when shoe was set. 1933.	Feet Depth Sunk Below Bottom of Open Trench. (Finished caissons are left 25 ft. above bottom of open trench.)					Maximum depth in feet reached below original ground-water.	Maximum lbs. per sq. in. air pressure at lowest point reached.	Date of sealing to ledge. 1934.	Average feet per week, including total time of construction sinking, pumping and sealing to ledge.	Underpinning Extended to Next Caisson.	
		Without Air Locks.		Additional Under Air.		Total to lowest point reached.					1933.	Air Pressure lb. per sq. in.
		To original ground-water level El. 416.	Below original ground-water level.	Until sinking stopped.	Under-pinning below shoe.							
West 31	Oct. 11	...	34	1	16	51	79	11	Jan. 26	3.3	.....	...
End												
1	Sept. 11	...	43	3	9	55	87	11	Jan. 15	3.0	.....	...
2	Sept. 7	...	35	12	19	66	101	14	Jan. 4	3.8	.....	...
3	Sept. 5	...	40	17	12	69	108	18	Jan. 12	3.7	.....	...
4	Aug. 30	...	36	25	7	68	111	18	Jan. 21	3.3	.....	...
5	July 28	...	23	39	12	74	119	12	Mar. 18	2.2	.....	...
6	July 26	...	28	42	6	76	120	18	Mar. 22	2.2	.....	...
7	July 21	...	36	34	9	79	125	21	Feb. 15	2.7	.....	...
8	July 19	...	24	47	13	84	130	20	Mar. 24	2.4	.....	...
9	July 18	...	38	34	11	83	130	19	Mar. 15	2.4	.....	...
10	July 15	...	38	34	9	81	128	25	Feb. 23	2.6	.....	...

10	July 15	...	38	34	9	81	128	25	Feb. 23	2.6	.....	...
11	July 12	...	43	29	12	84	130	21	Feb. 4	2.8	.....	...
12	Aug. 29	...	53	24	10	87	128	24	Jan. 11	4.5	.....	...
13	Exploratory	Caisson	..	..	..	..	(127)	(48)	1933	..	.....	...
14	May 22	...	60	21	12	93	125	17	Sept. 21	5.3	.....	...
15	June 10	...	69	16	10	95	121	17	Oct. 14	5.2	.....	...
16	May 19	...	60	31	5	96	116	22	July 22	10.3	.....	...
17	May 31	...	71	19	11	101	116	16	Nov. 8	4.4	.....	...
18	Mar. 22	...	48	42	15	105	115	11	Nov. 27	2.9	.....	...
19	June 23	...	81	8	13	102	105	4	Dec. 6	4.3	.....	...
20	Apr. 26	2	69	14	10	95	93	11	June 3	17.0	Dec. 9	7
21	June 5	7	72	0	12	91	84	10	July 27	12.1	.....	...
22	Apr. 26	13	61	4	8	86	73	11	June 3	15.4	July 29	11
23	June 12	19	53	6	7	85	66	6	Aug. 5	10.8	.....	...
24	Apr. 19	24	52	4	7	87	63	13	June 10	11.5	Aug. 7	7
25	May 27	25	48	3	8	84	59	8	July 8	13.7	.....	...
26	Apr. 27	25	38	0	19	82	57	13	June 15	11.5	Aug. 11	7
27	May 22	25	25	2	13	65	40	7	Aug. 16	5.2	.....	...
28	Apr. 27	26	11	4	17	58	32	11	May 26	13.6	Aug. 24	5
East 29	July 14	26	3	2	10	41	15	6	Sept. 2	5.6	.....	...
End 30	July 10	12	0	(No Air)	14	26	10	0	Oct. 13	1.9	.....	...

In a contract of this character the method of payment is very important. A price of ten cents per million foot-gallons for the pumpage was fixed before bids were opened, and the prospective contractors bid unit prices for the excavation to fixed levels, the measurement being regardless of whether the pumpage had made it possible to excavate in the open or not. Two unit prices were bid for concrete, one for that placed in the open and one for that placed under air. It is believed that by this arrangement the preliminary experimental work was capitalized to the fullest extent and the contractor was encouraged to perform the work in the most economical manner.

The principal items of the contractor's plant are:

3 Low-pressure air compressors with a total capacity of 2 550 c.f.m.
1 High-pressure air compressor with a total capacity of 500 c.f.m.
32 Feet height of steel caisson forms.
4 Muck locks, 2 man locks.
10 Ground-water lowering pumps, each 800 g.p.m., at 90-ft. head.

Upon the completion of the caisson wall the wells in the caissons and the open trench in and around the projecting tops of the caissons will be backfilled with impervious soil to an elevation somewhat higher than the brook bed. The entire area between the toes of the future dike will be left in readiness for a contract for the construction of the dike embankment.

PART IV

Costs and Miscellaneous Data.

In the following are presented miscellaneous data on costs, and other data on the Metropolitan Water District not readily found elsewhere.

Table 11.—Contracts for the New Supply.

No.	Description.	Date.	Contractor.	Cost
1	Borings on Tunnel Line	Nov. 15, 1926	Pennsylvania Drilling Co.	$18 641.69
2	Borings at Dam and Dike	Nov. 12, 1926	Sprague & Henwood, Inc.	26 885.83
3	Aerial Photographs	Nov. 22, 1926	Fairchild Aerial Surveys, Inc.	5 997.50
4	Shaft 5 and 1 367 ft. Tunnel Excavation	Mar. 30, 1927	James J. Coughlan & Sons, Inc.	290 581.99
5	Cordaville Pipe Line	Apr. 22, 1927	Lock Joint Pipe Co.	83 762.86
6	Cordaville Pumping Station	May 12, 1927	F. A. Mazzur Co., Inc.	17 385.04
7	Hopkinton Pipe Line	May 7, 1927	Cenedella & Co.	134 365.72
8	Shafts 6 and 7 and 3 062 ft. Tunnel Excavation	Jun. 2, 1927	James J. Coughlan & Sons, Inc.	456 784.26
9	Aerial Photographs	Mar. 30, 1927	Fairchild Aerial Surveys, Inc.	2 560.00
10	Transmission Line for Construction of Wachusett-Coldbrook Tunnel	Apr. 27, 1927	New England Power Co.	Construction 86 439.58 Maintenance 33 107.81
11	Ashland Pipe Line	Jun. 8, 1927	Antonio Mogavero	130 145.35
12	Shafts 2, 3 and 4 and 3 810 ft. Tunnel Excavation	Jun. 1, 1927	Dravo Contracting Co.	632 727.44
13	Whitehall Pipe Line	Jul. 20, 1927	Portland Contracting Co., Inc.	60 603.17
14	East Portion Wachusett–Coldbrook Tunnel	Apr. 30, 1928	West Construction Co.	3 515 787.08
16	Whitehall Open Channel	Sep. 28, 1927	Sanders Eng. Co.	32 695.16
17	West Portion Wachusett-Coldbrook Tunnel	Apr. 30, 1928	West Construction Co.	4 127 094.19
18	Iron Castings for Ware Intake Works	Jan. 14, 1929	Barbour Stockwell Co.	19 431.07
19	Dam and Substructure, Ware Intake Building	Jul. 23, 1929	J. W. Bishop Co.	285 580.49
20	Coldbrook-Swift Tunnel	Apr. 23, 1931	Wenzel & Henock Construction Co.	4 978 031.80*
21	5.2 miles Ware-Belchertown Highway	Jul. 20, 1931	Lane Construction In addition to Dept. Public Works' share	129 980.25 95 726.94
22	Unwatering Pump at Shaft 1	Nov. 14, 1929	F. A. Mazzur Co., Inc.	13 371.76
23	Borings at Dam and Dike	Nov. 13, 1929	Sprague & Henwood, Inc.	47 500.00†
24	Superstructure, Ware Intake Building	Jul. 23, 1930	Munroe & Westcott, Inc.	70 114.04
25	Superstructure, Wachusett Outlet Building	Jul. 23, 1930	Munroe & Westcott, Inc.	62 329.23
26	Wiring and Electrical Equipment, Wachusett Outlet Building	Sep. 24, 1930	Anderson-Coffey Co., Inc.	10 685.21
27	Covering Swamps along Ware River	Sep. 23, 1930	Raymond A. Bergesson	4 395.51
28	Grading Grounds, Ware Intake Building	Aug. 17, 1931	Anthony Ross & Son	8 164.14

* Basis of award, contract not yet completed.
† Estimated, contract not yet completed.

No.	Description.	Date.	Contractor.	Cost.
29	Grading Grounds, Wachusett Outlet Building	Aug. 17, 1931	Anthony Ross & Son	11 852.68
30	Stream Control Works, Main Dam	Aug. 1, 1931	Northern States Contracting Co.	544 610.60
31	Head House, Shaft 4	Aug. 26, 1931	E. D. Ward Co.	5 689.02
32	Exploratory Caissons, Main Dam and Dike	Sep. 16, 1931	Dravo Contracting Co.	162 838.33
34	Driveways, Quabbin Park Cemetery	Oct. 29, 1931	Lane Construction Corp.	7 842.94
35	Electric Power for Pumping at Dam Site	Jun. 2, 1932	New England Power Co.	Scheduled Rates
36	Caisson Core Wall, Dike	Dec. 28, 1932	West Construction Co.	999 265.00*
37	Electric Power for Pumping at Dike Site	Sep. 21, 1932	New England Power Co.	Scheduled Rates
39	Rutland-Holden Sewer	Jul. 11, 1933	James J. Coughlan & Sons, Inc.	184 327.50*
40	Transmission Line Relocation	Jun. 20, 1933	New England Power Co.	Construction 104 790.10 Damages 50 000.00
41	9.1 miles Belchertown-Pelham Highway (including Department Public Works' share)	Jul. 20, 1933	Carlo Bianchi Co., Inc.	222 335.50*
42	Sewage Disposal Agreement with City of Worcester	May 1, 1933		325 000.00
43	7.6 miles Extension of Belchertown-Pelham Highway to New Salem Center (including Department of Public Works' share)	Aug. 25, 1933	V. Barletta Co.	300 397.50*
44	4.1 miles Extension of Belchertown-Pelham Highway North of New Salem (including Department of Public Works' share)	Nov. 15, 1933	John Iafolla Construction Co.	179 480.50*
45	Exploration of Dam and Dike, Borrow Areas	Feb. 7, 1934	B. Perini & Sons, Inc.	6 200.00*

In general it is believed that payments to contractors on the work have included a fair profit to them.

* Basis of award, contract not yet completed.

TABLE 12.— SELECTED UNIT BID PRICES.

Description.	Unit.	Contract Price in Dollars, followed by Contract Number in Parentheses.
Borings		
Wash borings, 2½-in. extra heavy pipe	lin. ft.	2.50(1) 4.35(2)
Core borings, generally 20 ft. in sound bedrock, 1-in. diamond drill core:		
Surface to sound bedrock	lin. ft.	3.50(1) 4.35(2)
In sound bedrock, including boulders over 2 ft.	lin. ft.	4.50(1) 4.35(2)
Casing ordered left in place	lin. ft.	0.80(1, 2)
Hard Rock Tunnel		
Earth excavation for tunnel shafts	cu. yd.	6.50(4, 8) 10.00(12) 8.00(14) 7.00(20)
Rock excavation in shafts, and tunnel within 50 ft. of shafts	cu. yd.	14.90(4) 13.65, 14.00(8) 13.00, 13.50(12) 12.00(14, 17) 14.00(20)
Excavation in aqueduct tunnel except within 50 ft. of shaft	cu. yd.	10.00, 13.00, 13.50(12) 10.75(14) 11.00(17) 8.80(20)
Rock excavation in stream-control tunnel	cu. yd.	4.00(30)
Allowance for drainage in shafts and aqueduct tunnel	lin. ft.	22.20(4) 9.75(8) 15.00(12) 2.00(14, 17) 3.00(20)
Forms for shaft lining	lin. ft.	15.75(4, 8, 14, 17) 20.00, 22.00, 30.00(12) 15.00(20)
Concrete lining of construction shafts, except cement and grouting	cu. yd.	11.55(4, 8) 15.00, 16.00, 17.20(12) 8.00(20)
Concrete in special construction at shafts, except cement and reinforcement	cu. yd.	10.00(14, 17) 11.00(20) 6.80(30)
Forms for aqueduct tunnel lining	lin. ft.	2.60(14) 3.50(17) 1.11(20)
Concrete lining of aqueduct tunnel, except forms, cement and grouting	cu. yd.	7.50(14) 8.00(17) 3.00(20)
Concrete lining of stream-control tunnel and cut and cover section, except cement and grouting	cu. yd.	5.45(30)
Portland cement	bbl.	2.60(4, 17) 2.70(8) 3.20(12) 2.65(14) 2.15(20) 2.50(30) 2.70(36)
Permanent timbering in tunnel	M ft. B.M.	35.00(14) 25.00(17) 45.00(20) 100.00(30)
Temporary timbering in tunnel	M ft. B.M.	100.00(14, 17) 45.00(20)
Structural-steel roof-support in tunnel	lb.	.06(30)
Gunite protective coating of aqueduct tunnel rock roof	lin. ft.	2.50(20)
Gunite protective coating of stream-control tunnel rock roof	lin. ft.	12.00(30)

TABLE 12.—SELECTED UNIT BID PRICES—(Continued).

Description.	Unit.	Contract Price in Dollars, followed by Contract Number in Parentheses.
Open Cut Excavation and Embankment (Earth includes boulders up to ½ cu. yd.)		
Miscellaneous earth excavation	cu. yd.	1.00(14) 2.00(17) 1.50(20)
Earth excavation in large open cuts	cu. yd.	.21(30) .19, .20(36)
Removal of soil	cu. yd.	1.00(14, 20) .53(30) .25(34)
Stripping top 2 ft.	cu. yd.	.20(36)
Miscellaneous rock excavation	cu. yd.	4.00(14, 17) 6.00(19, 20)
Rock excavation from large portal cuts	cu. yd.	2.00(30)
Rock excavation, unrestricted blasting not permitted	cu. yd.	8.00(36)
Consolidated refilling and embanking	cu. yd.	1.00(14, 17) .50(20) .13(30) .12(36)
Unconsolidated refilling and embanking	cu. yd.	.10(30, 36)
Surface dressing of top soil	cu. yd.	1.00(14) 1.50(20)
Impervious filling in deep caisson wells	cu. yd.	1.00(36)
Rolled impervious embanking	cu. yd.	.40(30) .36(36)
Loam dressing	cu. yd.	.55(28, 29) .60(34) 1.00(39)
Seeding and grassing	acre	500.00(14, 20, 28, 29) 250.00(34) 200.00(39)
Masonry, Stone, Cement		
Miscellaneous concrete except reinforcement and cement	cu. yd.	15.00(14) 10.00(17) 12.00(19) 7.20(30) 7.50(39)
Steel for reinforcing concrete	lb.	.08(14) .05(17, 20) .045(19) .04(30) .03(39)
Portland cement	bbl.	2.65(14) 2.60(17) 2.15(20) 2.50(30) 2.70(36) 2.50(39)
Ashlar, except copings, dimension stone, and cement for joints	cu. yd.	35.00(14) 65.00(19)
Ashlar copings, except cement for joints	cu. yd.	50.00(14, 17)
Dimension stone	cu. yd.	165.00 (19)
Miscellaneous stone masonry, except cement for joints	cu. yd.	60.00(20)
Stone paving	cu. yd.	5.00(14, 17) 4.00(19) 3.00(20) 6.00(30) 3.00(39)
Concrete channel paving, except cement	cu. yd.	4.55(30)
Rip rap	cu. yd.	2.00(19) 1.00(20) 2.25(30) 2.50 (36) 2.00(39)
Crushed stone, gravel and sand	cu. yd.	3.00(14) 4.00(17) 3.50(19) 1.00(20) 2.00(30, 39)
Sinking Caissons (Contractor is also paid 10 cents per million feet-gallons for pumping to lower the ground water. Earth		

Concrete channel paving, except cement	cu. yd.	4.55(30)
Rip rap	cu. yd.	2.00(19) 1.00(20) 2.25(30) 2.50 (36) 2.00(39)
Crushed stone, gravel and sand	cu. yd.	3.00(14) 4.00(17) 3.50(19) 1.00(20) 2.00(30, 39)
Sinking Caissons (Contractor is also paid 10 cents per million feet-gallons for pumping to lower the ground water. Earth includes boulders up to ½ cu. yd.)		
Earth excavation in caissons from variable elevation of bottom of open trench to 106 ft. below original ground-water level	cu. yd.	5.85(36)
Rock excavation in caissons to 106-ft. level	cu. yd.	9.00(36)
Earth excavation in caissons below 106-ft. level	cu. yd.	13.00(36)
Rock excavation in caissons below 106-ft. level	cu. yd.	14.00(36)
Preparing ledge surface for seal to 106-ft. level	sq. yd.	2.00(36)
Preparing ledge surface for seal below 106-ft. level	sq. yd.	5.00(36)
Concrete including cutting edge, air lock wells, and reinforcement, except cement	cu. yd.	5.60(36)
Grouting rock seams, except drilling and cement	cu. yd.	7.00(36)
Portland cement	bbl.	2.70(36)
Pipe Lines		
Excavating and refilling pipe trench in earth	cu. yd.	1.25(5) 1.49(11)
Excavating and refilling pipe trench including ledge and boulders	cu. yd.	2.85, 4.90(7) 1.22, 2.40(13)
Excavating and refilling pipe trench in earth for 10-in. to 16-in. pipe	lin. ft.	.68(39)
Furnishing 30-in. reinforced concrete "Cylinder" pipe, including specials	lin. ft.	6.60(5)
Laying 30-in. reinforced concrete "Cylinder" pipe, including specials	lin. ft.	.45(5)
Gunite lined 48-in. steel pipe, embedded in tunnel lining	lin. ft.	13.00(30)
Furnishing 30-in. C. I. pipe, A. W. W. A., Class A, including specials	lin. ft.	6.00(7)

TABLE 12.—SELECTED UNIT BID PRICES—(Concluded).

Description.	Unit.	Contract Price in Dollars, followed by Contract Number in Parentheses.
Furnishing 24-in. C. I. pipe, A. W. W. A., Class A, including specials	lin. ft.	5.30(11)
Furnishing 20-in. C. I. pipe, A. W. W. A., Class A, including specials	lin. ft.	4.25(13)
Laying 30-in. C. I. pipe, A. W. W. A., Class A, including specials	lin. ft.	3.00(7)
Laying 24-in. C. I. pipe, A. W. W. A., Class A, including specials	lin. ft.	.65(11)
Laying 20-in. C. I. pipe, A. W. W. A., Class A, including specials	lin. ft.	.92(13)
Furnishing and Laying 16-in. C. I. pipe, Sandspun	lin. ft.	2.30(39)
Furnishing and Laying 12-in. C. I. pipe, Sandspun	lin. ft.	1.60(39)
Furnishing and Laying 10-in. C. I. pipe, Sandspun	lin. ft.	1.25(39)
State Highway Construction		
Roadway earth excavation	cu. yd.	.45(21) .19(41) .20(43) .27(44)
Ledge excavation	cu. yd.	.45(21) 1.00(41, 44) 1.15(43)
Ordinary borrow	cu. yd.	.15(21) .19(41) .10(43) .20(44)
Broken stone	ton	2.50(21) 1.40(41) 1.55(43) 1.80(44)
Bituminous material	gal.	.08(21) .105(41, 43) .09(44)
Gravel borrow	cu. yd.	.40(21) .24(41) .30(43, 44)
Sand borrow	cu. yd.	.50(21) .60(41) 1.00(43) .30(44)
Loam borrow	cu. yd.	.80(41, 44) .60(43)
Fine grading, rolling and finishing	sq. yd.	.03(21) .01(41, 43) .005(44)
Cobble guard rail	lin. ft.	.40(21) .35(41) .36(43) .33(44)
Miscellaneous		
Cast-iron shaft lining plates	ton	63.70(18)

It is difficult to estimate from bid unit prices alone, the total amount actually paid for certain units of the work. There follows a summary of selected average prices paid:

Aqueduct Tunnel. The total amount paid for sinking and lining the 14-ft. diameter *construction shafts* ready for tunnel excavation was $192 per foot for the aggregate depth of 2 608 ft. sunk under the preliminary shaft contracts, Nos. 4, 8 and 12, and $150 per foot for the aggregate depth of 883 ft. sunk under the tunnel contract, No. 20. The amount paid for excavating tunnel under the shaft contracts, without shaft head frames and regular tunnel excavating equipment, averaged $107 per linear foot.

Under Contract 20 the price paid for permanent timbering per foot of timbered tunnel was $9.30 for the timber in place and at least $6.10 for the additional excavation necessitated by the timbering. The amounts paid for *excavating and lining the tunnel* under the tunnel contracts were as follows:

	Per Linear Foot	
	Contracts 14 and 17	Contract 20
Net length between the shaft bottoms — ft.	66 512	54 431
Excavation	$76.80	$63.30
Permanent and temporary timbering and Gunite treatment	.10	.65
Distributed cost of the transmission line under Contract 10	1.60	..
Total length lined, or to be lined — ft.	74 870	54 862
Concrete lining, except cement	$19.80	$7.60
Cement	7.95	6.50
Miscellaneous items of sanitary, medical, hospital and other services, cleaning up, etc.	.45	.45
Total	$106.70	$78.50

Main Dam Stream Control Tunnel, Contract No. 30. Earth excavated from the approach cuts was used to construct the cofferdam embankments or toes of the future main dam embankment, and the amounts paid for such work are stated in the bid prices. As for the tunnel itself, *rock* excavation included the two portal cuts, the shaft, and the tunnel bore 955 ft. long. The upstream portal was carried 100 ft. farther into the ledge than would have been necessary if it had not been for the softness of the rock near the ledge surface at this point. For this excavation $198 390 were paid which is $208 per linear foot of tunnel bore or $3.26 per cubic yard of ledge in the bore and the portal cuts. In addition $223 735 were paid for concrete in the *lining* of the tunnel and shaft, the 275-ft. cut-and-cover extension of the tunnel bore, and the portal wing walls, including embedded pipes, stop-log grooves, structural steel in the closure piers, etc. This is $182 per linear foot of the completed structure between the entrance and exit faces, or about $10.80 per cubic yard of all concrete placed downstream from the approach channel. In addition $8 870 were paid for miscellaneous items of sanitary

and other services, river control contingencies, cleaning up, etc., so that the total construction cost, excluding the net cost of the earth-handling items, was $430 993 or $350 per linear foot of the total length of 1 230 ft.

Sinking Caissons, Contract No. 36. The total amount estimated for payment to date for sinking caissons, underpinning to ledge and sealing with concrete sufficiently to allow the air pressure to be taken off, in preparation for but not including drilling and grouting, also for pumping, concrete required for caisson projections 25 ft. above the bottom of the open-cut trench and for core wall completed at east end, and excavating and filling the end joints, was $647 812. This includes all the caisson sinking required in the dike core-wall and about 80 per cent. of the total excavation and filling of end joints required. The total amount of work for which this total sum was paid is divided as follows:

	Vertical Core Wall Area, Square Feet	Volume in Specified 9-ft. Width, Cubic Yards
Between stated levels:		
Above the original ground-water level	8 600	2 865
Between the original ground-water level and the specified payment line 106 ft. down	82 850	27 620
Below the 106-ft. depth	9 200	3 065
Total	100 650	33 550
With and without air pressure:		
Open caisson excavation (above the level to which the lowered ground water was encountered and the use of air locks necessitated.)	67 250	22 420
Pressure caisson excavation:		
Above the 106-ft. level	24 200	8 065
Below the 106-ft. level	9 200	3 065
Total	100 650	33 550
Total cubic yards of concrete in caisson projections above bottom of open-cut trench	..	11 550

Miscellaneous Heavy Equipment. The following items of equipment were purchased directly by the Commission for installation by its contractors:

20-ton hand operated travelling crane with one 5-ton electric hoist, installed in Wachusett outlet building under Contract No. 25. Span 32 ft. 9⅞ in. center to center crane rails, 1930. $3 534 delivered.

7½-ton electric travelling crane. Installed in Ware River intake building under Contract No. 24. Span 58 ft. 9⅞ in. center to center crane rails, 1930. $3 850 delivered.

4 Dow-disc butterfly valves, one 72-in. and three 60-in. including operating and stand-by oil pumping units, control equipment and all auxilliary apparatus, 1930. $28 267

220-ft. deep-well motor-driven pump, 4¾ in. by 24 in., 110 g.p.m., except the well casing in the tunnel dry-well at Shaft 1, 1929. $1 700 delivered.

The 900-h.p. motor-driven tunnel-unwatering pump and auxilliary equipment at Shaft 1 was not purchased separately by the Commission but

was furnished and installed under Contract No. 22. Also the 250-h.p. motor-driven pump in the Cordaville Pumping Station was furnished and installed under Contract No. 6.

TABLE 13.— EXPENDITURES FOR WATER SUPPLY WORKS.

Spent for the Wachusett-Sudbury-Cochituate Supply, and the Distribution Works.		
40-year sinking-fund bonds, issued 1895 to 1910		$41 398 000
30 to 40-year serial bonds, issued 1911 to 1925		4 287 000
Expenditure of current funds 1909, 1925 to 1934, including $300 000 requested appropriation for 1934		4 245 000
		$49 930 000
Paid for, to date.		
Adjustment of sinking fund, January 1, 1903	$2 256 804	
Assessments paid into sinking fund, 1903 to 1934	9 634 081	
Fees, net earnings of Wachusett and Sudbury plants, etc., paid into sinking fund 1903 to 1933 (to Nov. 30)	2 669 459	
Accumulated interest on sinking fund investments, 1903 to 1934 (including miscellaneous earnings since Nov. 30, 1933)	18 633 242	
Total sinking fund, April 1, 1934	$33 193 586	
Assessments to pay serial-bond annuities, 1911 to 1934	1 656 000	
Entrance fees, applied to construction, 1909, 1925	490 000	
Assessments to "pay-as-you-go," 1926 to 1934	3 755 000	
		39 094 586
Net water debt, April 1, 1934, account of Old Supply, after deducting $114 000, 1934 assessment to pay serial bond annuities due Jan. 1, 1935		$10 835 414
Spent for the New Ware-Swift Supply. (This includes both the cost of the Southern Sudbury Emergency Supply and a credit for the transfer to Worcester of rights in the Wachusett watershed.)		
20-year serial bonds, issued 1927		$1 000 000
30-year serial bonds, issued 1928 to 1934		27 500 000
Direct expenditure, 1927		1 000 000
		$29 500 000
Paid for, to date.		
Assessments to pay serial-bond annuities, 1928 to 1934	$3 451 000	
Worcester's fee, applied to construction, 1927	1 000 000	
		4 451 000
Net water debt, April 1, 1934, account of New Supply, after deducting $339 000, 1934 assessment to pay serial-bond annuities due July 1		$25 049 000

Approximate Distribution of Expenditures for Water Supply Works	
Wachusett-Sudbury-Cochituate Supply and Distribution Works:	
Wachusett Reservoir System	$12 786 000
Wachusett Aqueduct	1 798 000
Sudbury Reservoir System	7 655 000
Sudbury Aqueduct	3 705 000
Weston Aqueduct and Reservoir	7 031 000
Cochituate System and Aqueduct	1 869 000
Chestnut Hill and Spot Pond reservoirs, pumping stations, distribution system, and miscellaneous	11 906 230
Share in Metropolitan District Commission Building	250 000
Mystic River System and other old works	1 500 000
Total capital expenditures, Nov. 30, 1933	$48 500 230
Unexpended balance, and miscellaneous adjustments in the capital account	1 129 770
Capital account Nov. 30, 1933, not including $300 000 requested appropriation for 1934	$49 630 000
New Ware-Swift Supply (Including Southern Sudbury Emergency Supply).	
Construction of Quabbin Reservoir:	
Contracts for main dam	$599 770
Contracts for dike	804 980
Contracts for highway relocation	296 030
Contracts for electric transmission-line relocation	104 790
Contracts and force account for Quabbin Park Cemetery and cemetery removals	43 090
Miscellaneous equipment and construction	11 480
Construction of Quabbin Aqueduct:	
Contracts for Wachusett-Coldbrook tunnel section including lining of Shafts 1 and 8	9 161 950
Contracts for Ware River intake works	363 860
Contracts for Wachusett outlet works	98 240
Contract for head house at Shaft 4	5 690
Miscellaneous equipment and construction	90 630
Contract for Coldbrook-Swift tunnel section including Swift intake substructures	3 910 080
Watershed protection, real estate and damages for Ware-Swift supply	7 803 610
Engineering, overhead and administration for Ware-Swift supply	2 815 940
Southern Sudbury Emergency Supply	638 530
Joint Board investigation of watersheds	2 000
Construction of Rutland-Holden sewer	96 180
Total capital expenditures to Mar. 31, 1934 (Less $361 720 miscellaneous receipts credited to construction fund, and not including $450 620, estimated for payment on contracts but not yet paid.)	$26 846 850
Unexpended balance of bond issues	2 653 150
	$29 500 000

TABLE 14. — METROPOLITAN WATER DISTRICT ANNUAL ASSESSMENTS.

(Proportioned to the member municipalities, one-third on valuation and two-thirds on consumption.)

Date of Assessment, April 1.	ASSESSMENT.					METHOD OF DETERMINING APPROXIMATE SHARE OF EACH MUNICIPALITY IN DOLLARS.
	For Maintenance.	For bonds, all interest, and pay-as-you-go expenditures.		Credit by fees received.	Total.	P = Population in previous year in thousands. V = Valuation in previous year in million dollars. C = Consumption in previous year in m.g.d.
		Construction of old supply and distribution works.	Construction of new Ware-Swift supply.			
1905	$328 624	$1 848 962	.....	.....	$2 177 586	2 178 × (0.2176V + 0.4699P)*
1910	380 242	1 917 546	.....	.....	2 297 788	2 298 × (0.1958V + 5.596C)
1915	417 623	1 746 707	.....	.....	2 164 330	2 164 × (0.1673V + 6.227C)
1920	746 853	1 771 431	.....	.....	2 518 284	2 518 × (0.1656V + 5.528C)
1925	741 180	2 186 236	.....	$400 000	2 527 416	2 609 × (0.1273V + 5.209C)**
1926	769 799	2 143 553	.....	.....	2 913 352	2 996 × (0.1217V + 5.035C)
1927	847 554	2 244 488	$1 000 000	1 000 000	3 092 042	3 192 × (0.1171V + 4.975C)
1928	886 454	2 480 132	82 570	.....	3 449 156	3 551 × (0.1133V + 4.876C)
1929	825 574	2 318 569	432 697	.....	3 576 840	3 692 × (0.1108V + 4.759C)
1930	892 908	2 246 158	821 193	.....	3 960 259	4 082 × (0.1101V + 4.703C)
1931	966 102	2 292 221	953 919	.....	4 212 242	4 343 × (0.1080V + 4.724C)
1932	926 934	2 273 459	1 247 973	.....	4 448 366	4 598 × (0.1083V + 4.776C)
1933	857 624	2 079 536	1 504 215	.....	4 441 375	4 603 × (0.1109V + 5.025C)
1934	.....	2 089 377	1 763 390	.....	.....	.....

* Boston's share in 1905 = approximately 2 178 × (0.6527V).

** Brookline's share in 1925 (before deducting credit for water supplied from local sources) = approximately 2 609 × (0.1232V + 5.043C).

TABLE 15.— GROWTH OF THE TWENTY CITIES AND TOWNS *Now* IN THE METROPOLITAN WATER DISTRICT.

(With the exception of Swampscott, admitted in 1909, and Brookline, admitted in 1925, all were in the original District or admitted prior to 1903.)

Year.	Population.	Gallons per capita.	Consumption — m.g.d.			
			"Pump basis" to 1903, "Meter Basis" 1904 to date.	Members taking their entire supply from the District.	Total water-supply demand on District sources, including sales outside the District, etc.	
					Av. Annual.	Maximum Month.
1850	226 540	....	.....	.....	.....	...
1860	321 189	....	.....	.....	.....	...
1870	389 625	....	.....	.....	.....	...
1880	508 884	....	.....	.....	.....	...
1890	674 535	....	.....	.....	.....	...
1895	781 594	90.6	70.82	.....	.....	...
1900	897 635	111.4	100.00	.....	.....	...
1905	975 077	125.3	122.13	116.08	117.96	137
1906	.....	122.9	122.83	117.03	118.17	128
1907	.....	126.8	129.88	123.67	124.81	147
1908	.....	125.1	131.27	124.92	126.07	141
1909	.....	116.5	124.89	119.12	119.84	130
1910	1 098 048	107.6	118.22	112.09	112.96	125
1911	.....	103.1	116.09	110.00	111.37	120
1912	.....	105.5	121.59	116.23	117.39	139
1913	.....	92.7	109.45	103.85	104.91	116
1914	.....	93.5	112.88	107.04	108.45	124
1915	1 234 790	87.1	107.52	101.94	103.13	110
1916	.....	90.1	112.28	106.34	107.46	113
1917	.....	92.4	116.23	110.03	111.36	122
1918	.....	107.5	136.33	129.76	131.28	162
1919	.....	99.6	127.39	120.59	121.81	131
1920	1 290 651	104.1	134.40	127.27	128.65	149
1921	.....	95	124.69	117.41	118.74	128
1922	.....	95	126.74	119.27	120.74	129
1923	.....	98.1	133.04	125.24	127.25	136
1924	.....	96	132.18	124.10	126.84	135
1925	1 398 702	97.6	136.52	128.35	134.50	144
1926	.....	97.8	138.65	130.18	134.60	141
1927	.....	98.2	141.15	132.49	136.18	141
1928	.....	99.1	144.44	135.69	139.59	146
1929	.....	99.1	146.51	137.04	142.27	150
1930	1 498 253	97.5	146.11	136.42	143.11	158
1931	.....	95	144.57	134.78	138.09	145
1932	.....	89	137.68	127.99	131.48	139
1933	.....	87	134.39	125.07	128.64	136
1934*	.....	.....	.....	137	.....	...

*3 months.

TABLE 16.— CAPACITY OF PRINCIPAL RESERVOIRS AND AQUEDUCTS, METROPOLITAN WATER SUPPLY.

	CAPACITY MIL. GAL.				
	Total including flashboards as ordinarily operated.	Available above gate sill.	Area — acres.	Approximate grade-line capacity — m.g.d.	Length — miles.
Quabbin Reservoir	415 000	405 000	25 000		
Quabbin Aqueduct				340	24.61
Wachusett Reservoir	67 000	57 000	4 135		
Wachusett Aqueduct				350	11.95
Sudbury Reservoir	7 250	6 000	1 292		
Weston Aqueduct				300*	
Weston Reservoir	270	200	60		13.42
Sudbury Basin No. 3	1 180	1 000	250		
Sudbury Aqueduct				100	17.38
Chestnut Hill Reservoir	....	690	123		
Spot Pond	....	1 790	309		..

* Only about half of this until 2nd and 3d pipes are added in the Sudbury River and Happy Hollow siphons.

The above table does not include Cochituate Lake and Aqueduct and the following reservoirs in the Southern Sudbury area: Whitehall, Hopkinton, Ashland and Sudbury Basins Nos. 1 and 2.

PART V.

ORIGIN OF THE NAME QUABBIN.*

The new reservoir is named for the old Indian chief Quabbin who stood beneath the great oak on the broad slope of what is now Quabbin Hill, across the river from the little village of Enfield, and looked north as far as the eye could see along broad flat valleys speckled with mirror ponds and dotted with rocky peaks. These valleys had furnished good fishing and hunting for his tribe, the Nipmucks, for untold generations before the first white settlers came from Springfield. They were forerunners of the white man's thirsty civilization which would one day dam the narrow outlet and create a lake of greater extent than the old chief had ever seen. Tribes of Nipmucks, Nipmugs or Nipnets, as they were sometimes called, fished and hunted over all this section of central Massachusetts long before the coming of the white man to America. The Indian name implies that the tribes dwelt in "fresh-water fishing places" or in "well watered" valleys. Standing on Quabbin Hill overlooking the village of Enfield nestled in the valley five hundred feet below and viewing the vast and quiet expanse of the river

* In preparing the following descriptive notes, I have depended largely upon researches by George W. Coffin, one of the engineering force.

valley that is lost in the haze to the north, one can easily believe the historians who tell us these Indians were a peaceful people, unlike the warlike Mohawks on the west and Narragansetts on the east.

Historians tell us that the name Quabbin is probably the Anglicized form of Ninequabin, Naniquabin or possibly Wabequabban, the prefix Nine, or Nani, signifying chief. Be that as it may, local tradition gives us the spelling Quabbin and associates the old chief with the towering hill and the general locality both of which bear his name. It was not the custom of the red man to name places after persons, either living or dead. This was undoubtedly done by early white settlers of the region although it is true that the name was used by later remnants of the earlier Indian tribes.

The first white men to view the valley were probably a few adventurous spirits from the trading post established about the middle of the 17th century on the Connecticut River where Springfield is now located. The first real settlers to homestead in the valley obtained their grants to "a tract of land lying at a place called Quabbin" as a reward for services rendered either by themselves or their fathers to Massachusetts Bay Province as soldiers in King Phillip's War, particularly in the defence of Fort Narragansett in 1675. In 1738, a plat of 18 079 acres of land was laid out in the Swift River Valley and confirmed by the Great and General Court as Narragansett Township No. 4, which was the "area called Quabbin". This tract was "bounded Eastwardly on Lambs Town," (Colonel Lamb and his company were established in what is now Hardwick), "Southwardly on Reads Land," (Read apparently had a prior grant in what is now Ware), "Westwardly partly on Equivalent Land" (Governor Saltonstall's heirs *et al.* had been granted an equivalent of land in what is now Prescott), "and partly on Salem new Town, Northwardly partly on Salem Town and partly on Whites Town" (apparently to the north, near New Salem). The quotation is from Chapter 241 of the Acts and Resolves of the Province of Massachusetts Bay, 1738. In 1743 Pelham was incorporated and bounded "easterly on a tract called Quabbin," being the tract previously granted as Narragansett Township No. 4, and including what is now Greenwich, Enfield and part of Dana. This entire tract was incorporated in 1754 as the town of Greenwich. In 1816 the southerly part of this area was set off as the town of Enfield.

Apparently the early settlers who received the land as a reward for fighting King Phillip did not appreciate the "well watered" plain as did the Nipmucks who were well satisfied with the fishing. They were more interested in well drained upland areas where the maize and tobacco would grow without too much attention. Upon their complaint to the legislators, they were awarded additional "equivalent land" west of the Connecticut River near Williamsburg, which accounts for the name "Quabbin 2nd Grant" on early maps in that vicinity.

The remnants of Chief Quabbin's followers gradually dwindled as the white man overran their lands. Every waterfall furnished the site for a

grist mill and many thriving communities dotted hill and plain in the middle of the last century. It was on Prescott hill that Daniel Shay drilled his men for "Shay's Rebellion." Following the Civil War the great cities along the seaboard and the larger rivers continued their growth at the expense of rural communities, and began to suck the life blood from these tiny villages in the Swift River Valley.

Today there is a feverish activity of steam shovels and concrete mixers in the valley, but soon these evidences of the white man's occupation will disappear. The water will slowly creep up the sides of the Prescott peninsula where the deer will again roam undisturbed and the "well watered" valley extending north between rocky island peaks will be a fitting happy hunting ground for old Chief Quabbin.

Conclusion.

Many interesting features of the work have of necessity been omitted from this paper. Such for example are many details of the design and methods of construction, methods used by the engineering force in real estate and topographic survey work, and miscellaneous public relations.

PART VI.

Bibliography.

Massachusetts House Documents.

Report of the Massachusetts State Board of Health upon a Metropolitan Water Supply, No. 500, 1895.

Report of the Joint Board Consisting of the State Department of Public Health and the Metropolitan District Commission Relative to Water Supply Needs and Resources of the Commonwealth, No. 1550, 1922.

Report of the Joint Standing Committee on Water Supply Relative to the Present and Prospective Water Supply Needs and Resources of the Metropolitan District and Other Localities of the Commonwealth, No. 1724, 1924.

Report of the Metropolitan Water Supply Investigating Commission, No. 900, 1926; also this Journal, March, 1926.

Report of the Metropolitan District Water Supply Commission upon the Use of the Southern Sudbury and Cochituate Sources of Water Supply for the Metropolitan District, the Sanitary Requirements of the Sudbury River below Dam No. 1, the Requirements of the Town of Framingham for a Water Supply from the District Sources and the Matter of Sewerage in the Town of Ashland, No. 221, 1927.

Special Report of the Department of Public Health and the Metropolitan District Water Supply Commission Relative to Preventing Pollution of Water Supplies of Worcester and the Metropolitan Water District, No. 402, 1932.

This Journal.

Proposed Extension of the Metropolitan Water Works. By X. H. Goodnough. June, 1922.

The Massachusetts Water Report. By Allen Hazen. March, 1926.

Distribution Pumping Stations of the Metropolitan Water Works. By A. O. Doane. September, 1927.

Diversion of Interstate Waters for Domestic Water Supply. Litigation and U. S. Supreme Court Decision in Case of Connecticut *vs.* Massachusetts. By Frank E. Winsor. September, 1931.

The Boston Water Supply. By Frank A. McInnes. March, 1932.

The Relation of the Metropolitan Water Works to its Member Communities. By David A. Heffernan. March, 1932.

The Design of Pipe Lines (Southern Sudbury Emergency Supply). By Karl R. Kennison. March, 1933.

Journal of the Boston Society of Civil Engineers.

Subsurface Explorations for the Location of the Wachusett-Swift Tunnel. By Frank E. Fahlquist. January, 1929.

Diamond Drill Borings for the Swift River Dams. By N. LeRoy Hammond. January, 1929.

The Metropolitan District Water Supply Tunnel from the Ware River to the Wachusett Reservoir. By Frank E. Winsor. November, 1932.

New Type Open-Nozzle for Measurement of Sewage Flow (Rutland-Holden Sewer). By Karl R. Kennison. January, 1934.

Proceedings of the American Society of Civil Engineers.

Development of the Boston Metropolitan District. By Frederic H. Fay. December, 1929.

Administration of the Boston Metropolitan District. By Davis B. Keniston. December, 1929.

Metropolitan Water Supply. By Frank E. Winsor. December, 1929.

Surveys for Swift River Reservoir of the Boston Metropolitan District Water Supply. By N. LeRoy Hammond. December, 1929.

Levelling for Tunnels and Reservoirs for the Boston Metropolitan District Water Supply. By Laroy G. Harris, December, 1929.

Civil Engineering.

Boston's New Water Supply. By Frank E. Winsor. June, 1934.

The Ware River Intake Works of Boston's New Metropolitan Water Supply. By Karl R. Kennison, August, 1934.

Water Works Engineering.

Many Unique Features in Ware River Water System. By Karl R. Kennison. September 23, 1931.

Temporary Stream Control Tunnel. By Karl R. Kennison. September 20, 1933.

Big Reservoir to Increase Boston Water Supply. By Albert S. Genaske. April 18, 1934.

The Explosives Engineer.

Boston Provides for Its Future Water Supply. By A. E. Forster. December, 1927.

Boston Pushes Its Water Program. By M. R. Budd.

Part I — Main Dam Stream Control Tunnel. July, 1932.

Part II — The Coldbrook-Swift Tunnel Section of Quabbin Aqueduct. August, 1932.

Contractors and Engineers Monthly.

The Wachusett-Coldbrook Tunnel. By Howard J. Sprott. May, 1929.

Engineering News Record.

Large Equipment Units Speed Tunnel Work (Stream Control Tunnel). July 28, 1932.

Constructing a 14-Mile Tunnel for Boston's New Water Supply. September 11, 1930.

Engineering Graphic.

Driving the Wachusett-Coldbrook Water Tunnel. By George H. Richey. November, 1929.

Massachusetts Public Documents.

Annual Reports of the Metropolitan District Water Supply Commission, No. 147; 1926 to date.

Geological Data Report of the Chief Engineer, No. 147; 1927, 1928 and 1929.

Annual Reports of the Metropolitan Water Board, No. 57; 1896 to 1901.

Annual Reports of the Metropolitan Water and Sewerage Board, No. 57; 1902 to 1919.

Annual Reports of the Metropolitan District Commission, No. 48; 1920 to date.

Annual Assessments for Interest, Sinking Funds, Serial Bonds and Maintenance of the Metropolitan Districts, No. 92.

DISCUSSION.

FRANK E. WINSOR.* The author has covered the subject so thoroughly as to leave little opportunity for either criticism or constructive additions to the paper. It may be of interest, however, to consider in further detail some subjects covered by the paper.

Subsurface Investigations. The subsurface investigations for the Quabbin Aqueduct fixed the line of the tunnel as shown upon the plans which have been presented. A straight line from Wachusett Reservoir at Oakdale to the nearest point of Quabbin Reservoir would have been slightly shorter than the line adopted, and a straight line from Wachusett Reservoir at Oakdale to a point much nearer the southern end of Quabbin Reservoir would have been preferable from the standpoint of taking water from a greater distance from the entering points of the principal tributary streams, thus securing the benefit of a longer time in storage. It was necessary, however, that the line of the tunnel should pass under the Ware River within the limits prescribed by the legislature which necessitated a location considerably north of the straight line just referred to.

At the Wachusett end of the tunnel, where it is expected that a power plant may later be installed, there was a choice of building the aqueduct at the level of Wachusett Reservoir in which case it would have been necessary to provide for a connection to the power plant from the aqueduct under a 135-ft. head (with a maximum possible head at times of maximum diversion from the Ware of about 175 ft.) or, as was done, depress the aqueduct and run it under pressure, bringing the flow into the reservoir upward through a vertical shaft to which the power plant will connect. The subsurface investigations showed the high ground near the Wachusett outlet to be merely a glacial deposit but showed rock near the ground surface at Shaft 1 and coupled with estimates of cost indicated clearly that the method adopted is preferable.

Although the possibility of the power development alone might not have justified putting the entire tunnel through to Wachusett Reservoir under the full head of Quabbin Reservoir, this was the most practicable solution of the problem of diverting the Ware and Quinapoxet flows westward to Quabbin Reservoir or eastward to Wachusett Reservoir at will.

Investigations for the location of Shaft 2, two miles west of Shaft 1, indicated the desirability of an angle at this point both to secure rock near the surface for the shaft location and also to provide for the possible subsequent diversion of the Quinapoxet River into the aqueduct, from which point it may flow either eastward into Wachusett Reservoir or westward and be stored in Quabbin Reservoir. From Shaft 2 to Shaft 8, a distance of 11½ miles, the aqueduct line is substantially straight. West of Shaft 8 the line originally investigated with an angle at the shaft encountered rock

*Chief Engineer, Metropolitan District Water Supply Commission, Boston, Mass.

near the Ware River downstream from the shaft so low as to indicate the probability that earth would be encountered at the tunnel grade. For this reason the tunnel line from the east was extended straight through Shaft 8 and an angle made on the north side of the river, thus avoiding the difficulties and dangers of soft-ground tunnelling.

Investigations of the valley of Muddy Brook, where Shaft 11 is located, led to the adoption of a line considerably further upstream than originally contemplated in order to insure sufficient rock cover for the tunnel roof. Between Shaft 11 and the reservoir a considerable length of the line was obscure as to depth of rock, and while the geologists were of the opinion that there would be sufficient rock cover, it was deemed desirable to supplement the diamond drill borings by further subsurface investigations which for purposes of economy and to save time were made for a distance of about ½-mile by electrical methods.

The entire 25 miles of tunnel, as has been stated in the paper, are now driven, and no heavy ground was encountered at any point, the timbered sections, less than 0.4-mile in length, being for the purpose of insuring safety of workmen and to provide slightly increased thickness of masonry where the rock encountered was blocky or showed instability.

Another notable feature was the economy effected by the use of Gunite to coat the rock roof in portions of the tunnel somewhat over two miles in length where the air slacking of seams caused the loosening and falling of rock. The necessity of enlarging and timbering the tunnel was thus avoided.

The legislation required that the Commission shall collect and publish in its reports such information as to the geology of the region as will be of value in connection with the geological history of the State. Very careful geological investigations were made in advance, and records have been kept during construction. Samples of the rock encountered have been permanently preserved and will be available for future study. This 25-mile cross-section forms a unique permanent record of the geology of Central Massachusetts. It is expected that a complete geological report will be printed at an early date.

The work of the geologist has been of prime importance not only on the location of the tunnel but on the investigations of the main dam, the dike and the highways which have been built or are under construction. Studies both of the underlying rock and of the overburden have been of great importance as predicating in advance the location and character of rock to be encountered, the availability of excavated materials for masonry construction and the character of materials, such as gravel, and stone ballast, needed for the building of highways, for embankments of the main dam and dike, and other constructions.

Quabbin Aqueduct Operation. After the Quabbin Reservoir is completed the hydraulic grade in the Quabbin Aqueduct will stand over 100 ft. above the tunnel arch, and the pressure will be carried directly by the rock as the tunnel lining is well provided with vent holes so located as to prevent

undue pressure coming upon the lining either when the tunnel is under pressure or when it is empty. The ground water will be higher than the maximum hydraulic grade in the aqueduct except for some 3 000 ft. at the Wachusett end. For 500 ft. at this end the tunnel is circular and, together with the shaft, grouted solidly against the rock. Should there be slight external leakage from the tunnel at this end, the leakage will flow directly into Wachusett Reservoir.

The Quabbin Aqueduct is unique in the number of functions which it performs. Water taken from the Ware River at Shaft 8 may be conveyed eastward into Wachusett Reservoir or westward into Quabbin Reservoir. Due to the high elevation of the Ware River, no storage is required on the Ware watershed as the tunnel is of sufficient size to take the highest flood flows, although it has been built no larger than necessary to carry the required flow from Quabbin Reservoir to Wachusett Reservoir. Provision may easily be made so that waters from the Quinapoxet River, a tributary of the Wachusett Reservoir, having a drainage area of 44 sq. miles, may be diverted into the tunnel at Shaft 2 and similarly carried either to Wachusett Reservoir or Quabbin Reservoir.

Quabbin Reservoir. This reservoir is materially larger than would ordinarily be designed for a drainage area of its size. It is an unusual natural basin, storing water at the elevation adopted to an average depth of over 50 ft., and the expense of building it at a lower elevation would not be greatly reduced. The reservoir stores the flow not only from its own drainage area of 186 sq. miles but is available for storage of the flow from 98 sq. miles of the Ware and 44 sq. miles of the Quinapoxet. It is an ideal basin both as to shape and as to character of the soil, which is almost uniformly sandy with negligible areas of swamp. It is probable that in the far distant future when and if additional water is needed for the Metropolitan District it may be obtained by connecting other sources to Quabbin Reservoir and storing the waters therein. The portions of the Ware and Swift drainage areas to be used for water supply are very sparsely populated and with the long-time storage provided in Quabbin and Wachusett reservoirs will furnish a supply of low color and of excellent quality.

FRANK E. FAHLQUIST.* Construction of the Quabbin Aqueduct Tunnel and investigations at the dam sites have contributed greatly toward a better understanding of the geology of Central Massachusetts. This additional knowledge is confined not only to the purely scientific aspect of the subject, but as well to the applied side especially as it relates to the several rock types differing widely in character and behavior. A brief discussion of the geology of the area, in relation to this project, may be presented under two headings: first, the geology of the topographic features and surface deposits, and second, the geology of the underlying rock formations.

Geology of Topographic Features and Surface Deposits. The topo-

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graphical forms and surface deposits of this area, as throughout New England, are the results of several geologic processes, among which is that of glaciation. The widespread deposits of sand, gravel, boulders, and clayey material are the products of glacial abrasion and deposition. Two principal types of deposits are easily recognized. One of these, sometimes designated by the engineer as hardpan and by the geologist as unmodified drift, is composed largely of fine-grained particles and larger rock fragments such as cobbles and boulders. The other type, known as modified glacial drift, consists of sand and gravel showing evidences of having been deposited by the aid of water. In general, the former deposits are found in the higher elevations, whereas the latter are found in the valleys.

Except for a few scattered outcrops the character of the underlying rock formations has been obscured by occurrence of this drift material. The unmodified drift varies from a foot or less to about thirty feet in thickness, although in some instances this may be greater. The modified drift has a greater range of thickness and in some sections has been deposited to a depth exceeding 175 ft. It was found helpful in the location studies for the tunnel to locate the approximate limits of these two principal types, especially the unmodified drift, as this contributed some information at least to the problem of obtaining a rock profile. Each of these two kinds of material will have a part in the construction of the two dams in the Swift River and Beaver Brook valleys. Core material for each of these dams may be obtained from the unmodified glacial drift areas nearby, and heavier and more pervious material from the modified drift areas, within the lower parts of the valleys.

The underlying rock topography in many instances is quite different from that of the ground surface, and the only means of obtaining definite information for locating the tunnel was by some method of subsurface exploration. An extensive program of investigations by core borings was carried out, the borings in general being concentrated at recognized critical areas, chiefly in the Quinapoxet River, Ware-Prince River and Muddy Brook valleys. A location different from that originally planned was adopted in all these valleys, in order to be sure that the tunnel would be excavated in rock for its entire length. In the Quinapoxet valley the tunnel was depressed about 200 ft., but in the Ware-Prince and Muddy Brook valleys its location was changed in plan in order to pass under these valley floors at points more favorable for obtaining a safe rock cover. This, as closely as could be determined, amounts to between 50 and 60 ft., and in excavating through these two sections the conditions encountered were as favorable as they would have been with a much greater rock cover.

Removal of the glacial drift material throughout this area would reveal a drainage pattern considerably different from that now present. For instance many of the present surface divides are the result of accumulation and dumping of glacial drift material and have no counterpart in the underlying rock floor. This is the case of the Swift River divide in Muddy

Brook valley, where profiles developed from borings indicate a large continuous rock valley, which at one time was part of a much younger Swift River system. It is likewise true of conditions in Beaver Brook valley, where one of the dams for the Quabbin Reservoir is being constructed.

Geology of Underlying Rock Formations. All the rocks penetrated by the tunnel are of crystalline character and may be divided into the two classifications of granites and schists. The former occur in three distinct belts, and have been designated as the Fitchburg, Hardwick, and Monson granites. They occupy about eight miles of the total length of tunnel. The Fitchburg and Monson formations are typical light-colored granites of average hardness. The Fitchburg granite contains masses of a more micaceous rock, which in general is slightly softer than the granite but in some instances harder. These included rocks are remnants of an older formation into which the Fitchburg granite was intruded. A darker and harder igneous rock, or diorite, occurs as beds or broad bands in the Monson granite. The Hardwick granite is somewhat different from the Fitchburg and Monson granites in that it is much darker, contains much more mica than average granite, and is slightly harder. This formation also contains beds of a softer micaceous schist.

The schists or laminated rocks, which occupy about seventeen miles of the belt traversed by the tunnel are divided into five formations. Although they are now highly crystalline, they originally constituted a thick series of sedimentary rocks consisting of sandstone and shale. These primary differences may now be recognized by changes in the mineral content from bed to bed. What was originally sandstone now appears as quartzite or quartz schist. Portions of these formations which were shale have been changed into schist containing a high percentage of mica and having very fine laminations. Hence the term mica-schist has been applied to three of these formations, and quartz schist and quartzite to the other two.

The strength and hardness of the schist rocks has been greatly increased by the addition of granitic material. These igneous intrusions have permeated the rocks on a grand scale, so that it is often impossible to recognize whether some portions have originated as true igneous rocks, or as sediments which later have been transformed by being soaked with granitic constituents. Igneous intrusions generally occur as streaks, bands, and dikes, forming an interlocking network. In one particular mica-schist formation, about twenty-five per cent. of the present bulk is estimated to have been added. In general, the schist rocks are somewhat softer than the granites and of these the mica schists are the softest.

Two diabase or trap-rock dikes were penetrated by the tunnel, one being about 275 ft. wide and the other about 100 ft. These have been traced for several miles, in scattered outcrops and appear to cut across this area in a nearly parallel northeasterly direction. In the tunnel they are about ten miles apart.

The structure, or inclination of the bedding in the schists and the foliation or grain of the granites are surprisingly simple, although their interpretation is difficult. The geologic section of the tunnel shows a wide belt of only slightly inclined schist rocks flanked by rocks having a somewhat steeper inclination, with scattered areas where the structure is vertical or nearly so. The slightly inclined bedding of the rocks in the tunnel adjacent to Shaft 4 and Shaft 9 is one of the contributary causes of excessive rock scaling following excavation, which however was successfully remedied by Guniting. One of the geologic surprises in the construction of this project is the absence of crushed or faulted zones requiring timber support. In tunnelling through several rock formations over such a great distance, the occurrence of such weak zones would normally be expected. Faulting and extreme crushing has occurred at many points, but this happened before the formations were subjected to igneous invasions. The injection of granitic material into these zones replaced the older and weaker rock with one which was stronger and more crystalline. Very little permanent timbering was required, and most of this was necessary because of excessive scaling, due to a combination of conditions, including air slacking, thin bedding and the nearly horizontal position of the strata. At one point, however, in the Monson granite near Shaft 12, the tunnel was excavated through a zone of crushed and altered rock. In this section, about 120 ft. long, the clayey material encountered had a tendency to move and develop pressures.

ROBERT SPURR WESTON.* Mr. Kennison has honored me with a request to discuss his paper. However, the paper is so complete, especially regarding the general design and methods of construction, and has been so well discussed by Mr. Winsor that any further elaboration of its subject matter would be superfluous. I am, however, much interested in the quality of the water which the present works has supplied and in estimating that which will be supplied by the new works, also in the probable effect of the combination of the new with the old upon the water drawn from the Metropolitan taps in future years.

In 1922, when the Joint Board presented its report to the Massachusetts Legislature (House Document 1550), it was estimated that the consumption by the District in 1930 would be 168.5 m.g.d. Owing to economic distress, the actual consumption in that year was 143.1 m.g.d., and in 1933, the consumption had fallen to 131.48 m.g.d. The normal increase in consumption has been temporarily arrested, largely due to a decrease in the industrial use of water. Therefore, it is difficult to predict when and to what degree it will return, or what the more distant rate of increase will be. From past experience, it may be expected that the rate of increase will for a time be increased after industrial recovery takes place.

The rate of consumption is important from a water quality standpoint because storage is and will be the chief method of purification and the one most depended upon. The degree of purification by storage is largely

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dependent upon the period of storage and the latter upon the rate of consumption.

In the future, storage will be provided in the Quabbin, Wachusett and Sudbury reservoirs, as well as in the distributing reservoirs of the system, like Weston and Chestnut Hill reservoirs and Spot Pond.

The Quabbin Reservoir will be an ideal one. The reservoir is deep; its catchment area has a sparse and declining population; the natural soil is low in organic matter, and the period of storage long. With a yield of 210 m.g.d., and a storage capacity of 415 000 m.g., its storage ratio* will be 5.49, which is equivalent to about 5½ years. This may be compared with a storage ratio of 1.57, equivalent to over 1.5 years, for the Wachusett Reservoir. Storage of the Swift River water alone for 6.6 years, which would be the average condition were no water diverted into the Quabbin Reservoir from the Ware, or storage in the Quabbin Reservoir of the combined Ware and Swift waters for 5.5 years, will effect a high degree of purification. Although there may be some growths of organisms for a few years, while the reservoir is being conditioned, the reservoir will afterwards discharge a nearly clear and colorless water and one as safe as storage alone can produce.

By and large, it is appearance, particularly the color, which is the best criterion for judging this water. The color of the water which will be diverted from the Ware River, is about 55 p.p.m., the color of the Swift River water which will be stored in the reservoir is about 35 p.p.m., or slightly lower than that now entering the Wachusett Reservoir. But the period of storage is so long that a reduction to a color below 2 p.p.m. may be confidently expected,— just such a reduction as occurs in some of the lakes and ponds within the Chicopee water-shed which provide equally long periods of storage. So long is the period of storage that it will make little difference whether or not the natural runoff of the Swift catchment area or the more highly colored diversion water from the Ware, or the somewhat polluted Quinapoxet water is stored. It will, however, make considerable difference in the color of the water discharged from the Wachusett Reservoir whether it is fed by the runoff from its own catchment area, by the flood waters of the Ware, or the stored waters from the Quabbin Reservoir. The Wachusett feeders discharge a water having a color of about 38 p.p.m., which, strangely enough, is but slightly higher than that of the Swift River water. The color of the diverted Ware is estimated at 55, and the stored Quabbin water at 2 p.p.m.

Notwithstanding the construction of a sewer to care for the pollution within the Ware watershed and the demolition of buildings on property purchased for protection, the Ware River at Coldbrook can never be free from pollution, although it may be reduced to a slight amount. It may be guessed, therefore, that whenever practicable, the management will store the Ware water with the Swift water in the Quabbin Reservoir, and after

*The storage ratio is the quotient of storage capacity divided by annual yield.

as long a period of storage as can be secured in practice, will let it down into the Wachusett, thus feeding the latter with the highest practicably obtainable grade of water. With the Wachusett, material pollution begins. The city of Worcester is overflowing into the valley of the Quinapoxet, one of the feeders. Furthermore, this river receives considerable amounts of industrial wastes. The town of West Boylston at the head of the reservoir is growing, and in summer thousands of motorists pass daily. The Stillwater, the other main feeder, is also somewhat polluted. So that although the Commission may do much to control conditions, and although as Mr.

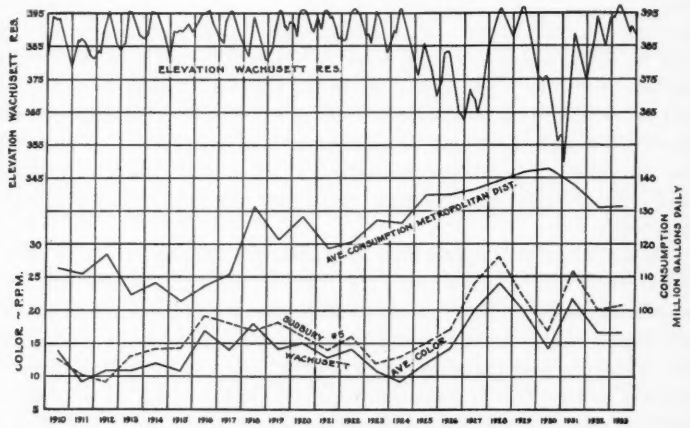

FIG. 20.— ELEVATIONS OF WACHUSETT RESERVOIR, WATER CONSUMPTION OF METROPOLITAN DISTRICT, AND COLORS AT WACHUSETT AND SUDBURY RESERVOIR DAMS, 1910–1933.

Winsor has stated, a part of the Quinapoxet may be diverted to Quabbin Reservoir and there purified by storage before reaching the Wachusett, pollution must be reckoned with.

Here storage again comes into play. The Wachusett stores 67 000 m.g. when full. With its mean annual runoff of 42 655 m.g. this is equivalent to a storage ratio of 1.57, equivalent to about 573 days. This period will effect an average reduction in color of at least 65 per cent. and render the water safe. Soon after the Wachusett was placed in service, long periods of storage obtained, and colors of delivered water were low. As the consumption increased, both period of storage and color decreased. This is shown in the following table and diagram (Fig. 20), giving the colors of the delivered water for various years.

The available catchment area of the Wachusett was formerly 108.8 sq. miles, but the new legislation on behalf of Worcester has reduced that which can be devoted to the Metropolitan District to 88 sq. miles. Conse-

AVERAGE COLORS OF WATER AT WACHUSETT AND SUDBURY DAMS.
(From Reports of the Metropolitan District Commission.)

Year.	ELEVATION OF WACHUSETT RESERVOIR.*		AVERAGE COLOR — p.p.m.		Total Consumption m.g.d.
	Maximal	Minimal†	Clinton Dam	Sudbury Dam	
1910	394.25	378.52	14	13	112.96
1911	387.41	381.3	9	10	111.37
1912	395.36	383.71	11	9	117.39
1913	395.38	386.5	11	13	104.91
1914	395.43	381.17	12	14	108.45
1915	392.42	389.0	11	14	103.13
1916	395.75	384.97	17	19	107.46
1917	395.55	382.5	14	18	111.36
1918	393.50	380.77	18	17	131.28
1919	395.93	387.53	14	18	121.81
1920	396.01	388.3	15	16	128.65
1921	396.06	386.42	13	14	118.74
1922	396.24	386.5	14	16	120.74
1923	395.65	381.42	11	12	127.25
1924	396.39	376.0	9	13	126.84
1925	386.19	369.60	12	15	134.50
1926	383.25	362.76	14	17	134.60
1927	385.44	365.1	20	24	136.18
1928	396.61	388.7	24	28	139.59
1929	396.50	373.5	20	22	142.27
1930	376.55	350.00	14	17	143.11
1931	388.79	374.66	22	26	138.09
1932	394.40	384.1	17	20	130.92
1933	396.55	387.81	..	..	131.48

*Elevation of high water — 395 ft.
†This is the minimal elevation either in the fall of the year or early in the following year.

quently the storage ratio under the new conditions is increased to 1.95, which is equivalent to about 710 days, assuming no addition from the new sources. It is probable, however, that Worcester will not require additional water for many years. Consequently this addition to the storage ratio need not be considered for some time.

The runoff from this area is inferior to the stored water from the West, although lower in color than the unstored Ware River water at Coldbrook. All of it, however, must be stored and used. It cannot be replaced by the new water.

Assuming a draft of 150 m.g.d. from Wachusett, the colors of the entering water are estimated at 41 p.p.m. with the unstored Ware being let down, or 28 p.p.m. with the stored Quabbin water being let down, and the result-

ing colors of delivered water at 18 and 12 p.p.m., respectively. Naturally, if the Quinapoxet is diverted through Shaft 2, stored in the Quabbin Reservoir and then let down again, the color and quality of both entering and delivered waters will be somewhat improved. The excellence of the discharge of the Quabbin Reservoir would still be maintained.

When 200 m.g.d. is supplied to the Wachusett Aqueduct with the Wachusett catchment area reduced to 88 sq. miles, and stored water let down from the Quabbin Reservoir, the color of the entering water may be reduced to about 19 p.p.m., and the color of that at the Wachusett intake to about 10 p.p.m. Further increments of stored water from the Quabbin Reservoir may reduce the entering color to below 15, perhaps to 13 p.p.m., but with reduced storage in the Wachusett at the higher rates of consumption, the color of the water at the intake of the Wachusett Aqueduct will rarely go below 8 p.p.m. which is, however, lower than it has ever averaged.

Insofar as color and bacterial contents go, the water in the Wachusett Aqueduct should be of high grade. Whether or not the reduction in storage in the Wachusett Reservoir to considerably less than a year's flow will promote the growth of micro-organisms, is difficult to predict. It may be possible that the smaller amount of organic matter in the Quabbin water will compensate for the reduced effects of storage in the older reservoir. If not, high numbers of microscopic organisms and consequent disagreeable odors and tastes may occur for short periods of the year as they have before.

With the Sudbury Reservoir, new problems arise. Located in a growing suburban zone, it is undoubtedly true that the Wachusett Aqueduct water will deteriorate in passing through it. At present, considerable trouble is experienced with growths of organisms in this reservoir, and the water now leaving it is somewhat inferior to that leaving the Wachusett Reservoir, particularly in appearance, which is the consumer's chief criterion. The Sudbury Reservoir stores but 7 253 m.g., and has a mean yield of but 22 m.g.d. With 120 m.g.d. passing through it, and assuming complete displacement, which, of course, cannot be obtained under the conditions, the period of storage is but sixty days, while with 300 m.g.d. passing through it, it will be but fourteen days. Experience has shown that reservoirs providing these short periods of storage usually suffer from growths of microscopic organisms. Furthermore, the arms of the reservoir, which are outside the current of water flowing through from the Southboro channel to the intake at the dam, will at times produce excessive growths of organisms, and these may seed the better water flowing through and promote growths not only near the dam, but in Weston Reservoir, Chestnut Hill Reservoir, Spot Pond and elsewhere. Treatment with copper sulphate, however, may control these growths sufficiently.

The engineers who have reported upon the general problem in the past, have all recommended the ultimate filtration of the supply, and some are agreed that a purification plant could well be located in Framingham, along the line of the Weston Aqueduct. This opinion is in accord with the

fundamental sanitary engineering principle, namely, to get as good a natural water as you can, and then purify it as best you can.

It is also possible, as Mr. Hazen has suggested, to construct a new distributing tunnel from the Wachusett Reservoir to the district, which would discharge a water which would require nothing more than disinfection to make the water wholesome and agreeable. This plan, of course, would leave the Sudbury sources unpurified, but it would of course cost far less to purify these alone. Furthermore the water of the South Sudbury now wasted could be included in the plan. While disinfection of the water from the Sudbury sources would make the water safe, it is also probable that the treated water would not meet esthetic standards, especially when compared with water supplied directly from the Wachusett Reservoir. It is also probable that the addition of treatment with copper sulphate would fail to make the water acceptable.

When this problem of appearance must be met, by filtration or otherwise, is difficult to estimate. Its solution may be forced by the water let down from the Quabbin Reservoir before the latter is aged, or by a reduced period of storage in the Sudbury Reservoir, both tending to increase odors and tastes. On the other hand, treatment with copper sulphate may tide over the initial difficulties, or long storage in Quabbin Reservoir may compensate for reduced storage in the Sudbury Reservoir, and the necessity for purification or a distributing tunnel may be postponed.

Various cities in the United States, for example New York, Cincinnati and Kansas City, have found that uncontaminated water is often recontaminated in passing through open reservoirs. Connected with one supply supervised by the writer's firm are two reservoirs,— one open and one covered, on similar hills, but both supplied from the same source. During 1933, intestinal bacteria (*Bact. coli*) were found in the water supplied from the open reservoir for over 8 per cent. of the time, while gas-forming bacteria were present seven months out of twelve, as compared with finding *Bact. coli* in the water from the covered reservoir less than 2 per cent. of the time, and gas-forming bacteria for only one month out of twelve.

Neither upland storage nor filtration will prevent accidental or vicious contamination of distributing reservoirs by birds and animals, including the human animal, and it may be found judicious to cover some of the smaller reservoirs in the Metropolitan District, although it will probably be more practicable, as at present, to depend upon disinfection with chlorine for protection against the accidental contamination of the larger ones.

Massachusetts may look with pride upon the works which have supplied and will supply Boston, from the Jamaica Pond works with its wooden pipe line, to the Quabbin Reservoir and connections which Mr. Kennison has so interestingly described. While many of us may not be active when the Quabbin water first reaches the local taps, we can be thankful for the good judgment of the various water-supply commissions, for the

careful planning, past and present, which has characterized the Metropolitan Works, and for the kind of engineers under whose direction the works have been built and operated so well.

ARTHUR D. WESTON.* Mr. Kennison's paper covers clearly and concisely the works constructed by the Metropolitan District Water Supply

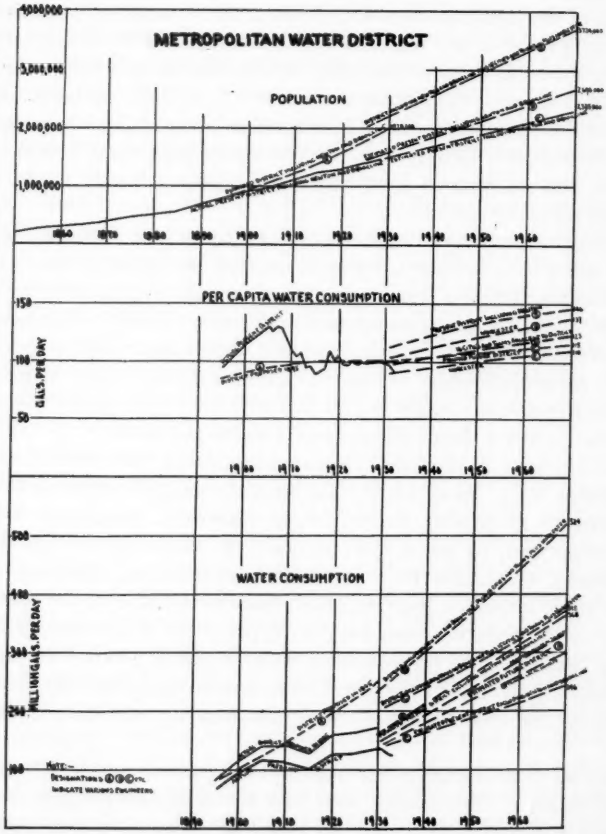

FIG. 21.— POPULATION AND WATER CONSUMPTION OF METROPOLITAN WATER DISTRICT.

Commission and should prove to be of considerable value not only to the members of this Association but also to the public authorities and those engaged in water-works practice.

With the completion of the tunnel from Wachusett Reservoir to the Ware River at Coldbrook, whereby water has already been diverted to

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supplement the water supply of the Metropolitan Water District, it may be of interest to make comparisons of the various estimates concerning the water supply needs and resources of this district and the contemplated and actual effect of such diversion on the quality of the water of Wachusett Reservoir.

Population. In discussing ways and means of increasing the water supply of the Metropolitan cities and towns, F. W. Stearns, as chief engineer of the then State Board of Health, estimated in his report published in February, 1895 as House Document No. 500 of that year, that a population of 1 978 000 would be reached in the year 1930 by the cities and towns now included in the Metropolitan Water District plus the city of Newton and the town of Brookline. The district failed to grow as rapidly as estimated by Stearns and the actual population of this group according to the census of 1930 was 1 498 253.

Water Consumption. Stearns estimated that the per capita water consumption in 1930 would equal 100 g.p.d., but extensive metering of services was necessary to reduce former high consumption to this figure by 1930. Goodnough in his report to the Joint Board, as published in House Document No. 1550 of the year 1922, and Metcalf in connection with the report of the Metropolitan Water Supply Investigation Commission, published as House Document No. 900 of the year 1926, starting with actual water consumptions estimated that the per capita use of water would increase until in the year 1970 it would reach 146 g.p.d. (Goodnough) and 115 g.p.d. (Metcalf). The estimated future water consumption from Metropolitan sources was estimated by Goodnough and Metcalf on various assumptions as to the municipalities which might enter the district and the degree to which they might continue to use their own sources; therefore these estimates may not be compared on the same basis.

As things have turned out, it looks as if, during the next ten years at least, the estimates of Metcalf, both as regards water consumption and population, will resemble more closely the actual conditions. The various estimates made in regard to the future water supply of the Metropolitan Water District are shown in Fig. 21, those of 1895 being marked A, those in relation to the 1922 study as B, and those of the 1926 report as C.

Drainage Areas and Yields. A comparison of the various ways and means proposed for securing an additional water supply for the Metropolitan Water District appears to the writer to be particularly interesting. The 1895 study under the direction of F. W. Stearns assumed that about 173 m.g.d. of water would be available from the Sudbury River, Lake Cochituate and Wachusett drainage areas, that additional water from the South Sudbury and Assabet River drainage areas would bring the safe yield to 201 m.g.d. and when necessary by diverting the Ware River at Coldbrook some 71 m.g.d. additional would be secured and finally, by utilizing the Swift River and Lower Ware River drainage areas for an additional 200 m.g.d., a total of 472 m.g.d. would be available. Compared with

these estimates, the Joint Board in 1922 proposed the development of the North Sudbury sources with Wachusett and the Ware River at Coldbrook to yield about 160 m.g.d. and the subsequent development of the Swift River by utilizing the flood flows of this river over 1.2 c.f.s. per sq. mile and the construction of a large reservoir which would bring up the safe yield of the sources to 233 m.g.d. This report also contemplated later development of the flood flows of the Millers River, when necessary, and possibly of the lower Ware River watershed, making a safe yield from all sources of about 352 m.g.d. It will be remembered that the investigating

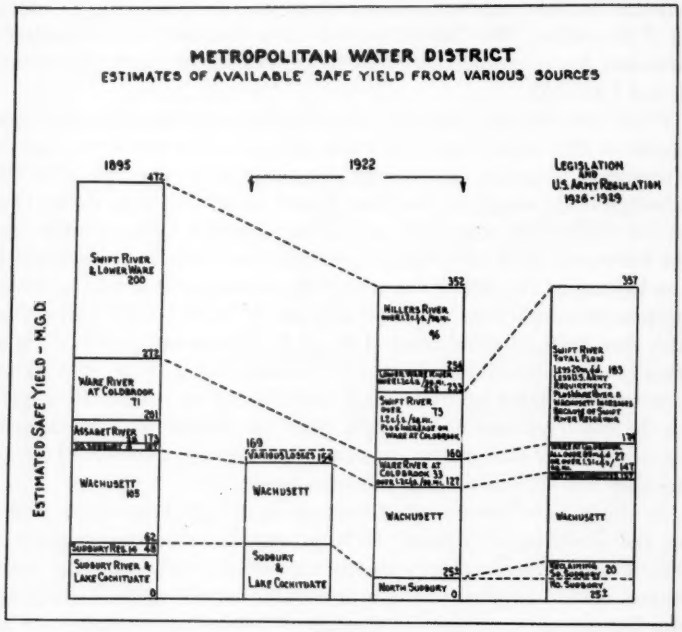

FIG. 22.

commission, known as the Gow Commission, in the report of 1926, recommended the development of sources of water supply chiefly in the easterly part of the state and, therefore, these estimates cannot be compared with those of 1895 and 1922.

After consideration of the recommendations of the Metropolitan Water Supply Investigating Commission and those of the Joint Board in 1922, legislation was enacted by the Commonwealth and later regulations were adopted by the U. S. Army. In accordance with this legislation and the U. S. Army regulations the present works were designed for a safe yield of the various sources as follows:

North Sudbury area 25 m.g.d.
South Sudbury area 20 m.g.d.
Wachusett drainage area and purchase from Worcester 102 m.g.d.
Development of Ware River at Coldbrook 27 m.g.d.
Flood flows from Swift River, additional diversion of the Ware River at Coldbrook 183 m.g.d.

The total yield is 357 m.g.d., which is about the same in gallons per day as proposed in 1922. A comparison of these various developments as proposed is shown in Fig. 22.

A comparison of the future estimated requirements of the Metropolitan Water Supply District with estimates of the safe yield of the sources

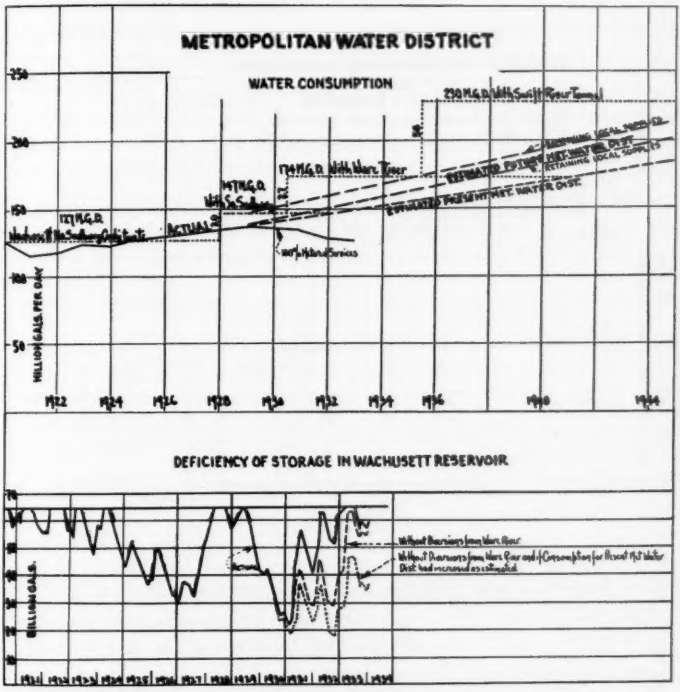

FIG. 23.— WATER CONSUMPTION AND DEFICIENCY OF STORAGE IN WACHUSETT RESERVOIR.

to be developed would indicate that Wachusett, North Sudbury and Lake Cochituate, plus the development of the South Sudbury, would have been sufficient up to 1931 if the use of water from Metropolitan sources had increased as estimated and if those municipalities which it was expected would enter the district had entered and continued to use their local sources. If membership in the district had increased as was contemplated

and these new members had not continued to maintain their local sources of supply, the safe yield probably would have been equalled by 1929. Actually, because of decreased water consumption, the safe yield of this supply was sufficient for the Metropolitan Water District through 1931 and would have been sufficient under the actual yields obtained even without the Ware River. This condition, which is shown in Fig. 23, is particularly interesting upon analysis when studied together with the depletion of storage of the sources of water supply as shown on the bottom of this diagram.

It should be remembered that in estimating the safe yield of the present sources of water supply the dry period from 1908 to 1915, inclusive, was used as a basis. This period probably is the longest dry period of that

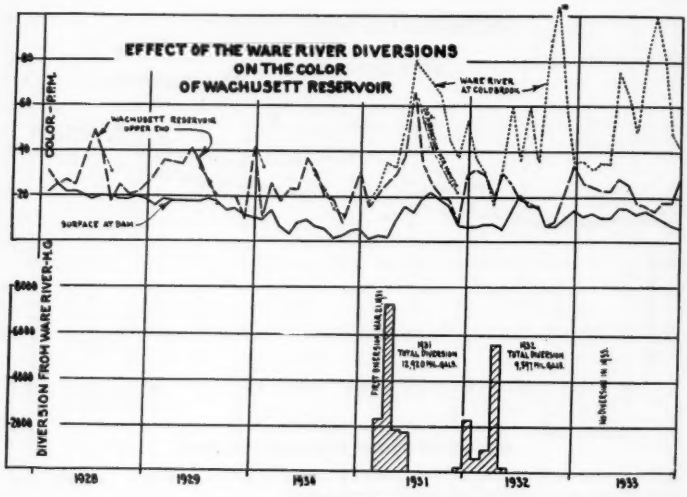

FIG. 24.

extent on record in this vicinity, but in the actual operation of the works with the water consumption so nearly equal to the safe yield, the short dry period of 1929 to 1931 was critical and by the middle of February, 1931 Wachusett was drawn to the low elevation of 45 ft. below the spillway.

Water Quality. While the District did not actually face a shortage of water, it did face a shortage of water of good quality, and the completion of the Ware River works made possible the replenishment of the supply at a time when it was most needed. Water was first diverted from the Ware River into Wachusett Reservoir on March 21, 1931, the storage in this reservoir having been depleted by some 44 billion gallons. Under these conditions, an excellent opportunity was afforded for studying the effect of the diversion of the water from the Ware River on the water in the reservoir. During the period from March 21, 1931, to the end of the year

12 920 900 000 gal. of water were diverted from the Ware River, and the color of this water varied from 33 to 80 p.p.m. The water of Wachusett Reservoir at the upper end, which was a mixture of water from its own tributaries with that discharged from the Ware River tunnel, had a color of 30 to 67 p.p.m. The effect of this diversion on the color of the water in Wachusett Reservoir is shown in Fig. 24.

A study of this diagram shows that the reduction of the color of the water in Wachusett Reservoir coincides with the curve referred to by Fair (using 20 per cent. per month reduction) in his discussion of a paper read by Kennison before this Association in 1928.*

JOHN L. HOWARD.† The Metropolitan Water Board acquired the supply works and pumping stations of the city of Boston January 1, 1898, and paid for them the sum of approximately $14 000 000. It also acquired Spot Pond from Malden, Medford and Melrose for the sum of $1 240 000. It constructed the Wachusett Dam, Aqueduct and Reservoir; completed the Sudbury Dam and Reservoir that was started by the city of Boston in 1894; laid about 165 miles of supply and distribution mains; built 4 pumping stations and an addition to the old high-service pumping station of the Boston Water Works; constructed the Weston Aqueduct and Reservoir; built two hydro-electric power stations and a 66 000-volt electric transmission line, 16 miles in length. Total construction expenditures amounted to about $48 000 000.

The value of sinking fund to retire bonds issued for construction purposes, prior to 1912, is approximately $32 000 000 and the net debt April 1, 1933 is about $12 500 000, excluding bonds issued for construction of the new works to the Ware and Swift rivers by the Metropolitan District Water Supply Commission. Since 1912 all bonds issued have been serial bonds, amounting to $4 300 000 and of these $1 300 000 have been paid. The storage reservoirs have a capacity of about 80 billion gallons. Four aqueducts have a total length of 56.5 miles. The Wachusett and Weston aqueducts have a capacity of 350 and 300 m.g.d. respectively, while the Sudbury Aqueduct has a capacity of 109 m.g.d. and the Cochituate Aqueduct 20 m.g.d. The distribution reservoirs have a capacity of about 2 400 m.g. and 3 standpipes a capacity of 4.83 m.g.

The operation of the hydro-electric stations over a period of 20 years produced a gross income of $1 300 000 and after deducting operating expenses and interest and sinking-fund requirement charges leaves a profit in 20 years of about $500 000, which has been used to reduce the assessments of the Metropolitan Water District.

The five pumping stations combined have a pumping capacity of 340 m.g.d. and a horse power of 7 500.

The Metropolitan Water District has an area of 174 sq. miles and an estimated population regularly supplied of 1 438 440 in 18 cities and towns.

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*This JOURNAL, Vol. XLIII, No. 1.

†Deputy Chief Engineer, Metropolitan District Commission, Boston, Mass.

The average consumption in 1933 was 125 074 000 g.p.d., equivalent to 87 g.p.d. per capita.

There follows a summary of information on the old works of the Metropolitan Water District.

Wachusett Dam and Power Plant.

Wachusett Dam:

Work began in 1900 and was completed in 1906.

Length of main dam, including abutments	971 ft.
Length of waste weir	452 ft.
Elevation of top of main dam	El. 415 ft.
Height to top of dam above full reservoir level	20 ft.
Height to top of dam above finished surface of ground below the dam	114 ft.
Maximum height above rock foundation, about	207 ft.
Maximum thickness of dam at bottom	185 ft.
Thickness of dam at full reservoir level	25 ft.
Maximum depth of water at dam	129 ft.
Masonry required for dam	280 000 cu. yd.
Amount of contract	$1 603 635
Contractor	McArthur Bros. Co., of Chicago, Ill.
In charge of work	Winston Brothers & Locher

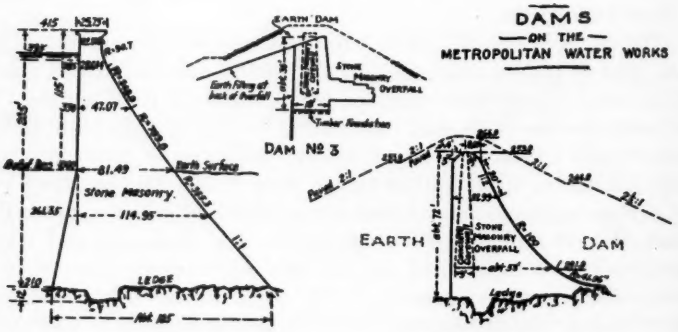

Fig. 25.

Power Plant:

Equipment installed in 1911.

S. Morgan Smith Co., of York, Pa., received contract for installing both hydraulic turbines and electric generators. Generators and other electrical equipment, furnished under a sub-contract by the Westinghouse Electric & Manufacturing Company of Pittsburgh, Pa.

4-48-in. C. I. pipes pass through the dam, inverts at El. 284 or 111 ft. below the high water in the reservoir.

Plant consists of 4-1 340 h.p. hydraulic turbine units of the spiral case, horizontal shaft type, each directly coupled to a 1 000 k.v.a., 60-cycle, 3-phase, 13 800 volt, alternating current generator, together with 1-90 h.p. and 1-110 g.p. horizontal shaft turbines,

each directly coupled to a 60-kilowatt direct current 125 volt generator used in exciting the main generators.

Speed of main generators is 400 r.p.m.

Diameter of penstocks ..... 48 in.
Diameter of wheels ..... 30 in.
Contract price of 4 complete units $71 500

Average head 97 to 98 ft.

2 lines of 3-conductor paper insulated lead-covered cables, 815 ft. long, composed of 19 strands of copper wires, tested to 30 000 volts, and laid in 3½-in. dia. Orangeburg fiber ducts.

Wachusett and Sudbury Reservoirs.

Wachusett Reservoir:

Work began in 1897 and was completed in 1905.

Reservoir first filled on May 10, 1908.

Area of watershed is 108.84 sq. miles.

Elevation of full reservoir above Boston City Base ..... 395 ft.
Water surface ..... 4 135 acres or 6.46 sq. miles
Land surface ..... 4 775 acres or 7.46 sq. miles
Full capacity ..... 64 968 000 000 gal.
Length ..... 8.41 miles
Maximum width ..... 2.05 miles
Maximum depth ..... 129 ft.
Average depth ..... 48 ft.
Length of shore not including islands ..... 37 miles
Soil removed from reservoir ..... 6 900 000 cu. yd.
Roads discontinued ..... 19.25 miles
New roads built ..... 11.8 miles

North Dike is about 2 miles long.

Maximum height above full reservoir ..... 17 ft.
Maximum width above full reservoir ..... 1 930 ft.
Amount of earth in North Dike ..... 5 161 814 cu. yd.

South Dike is about 2 800 ft. long.

Maximum height above full reservior ..... 19 ft.

Central Mass. R.R. — 6½ miles discontinued — 4½ miles new location — steel viaduct across the river is 917 ft. long with a maximum height above the river bed of 133 ft.

Sudbury Reservoir:

Work began in 1894 by the city of Boston and was completed by the Metropolitan Water Board in 1898.

Elevation of full reservoir above Boston City Base ..... 260 ft.
Water surface ..... 1 292 acres or 2.00 sq. miles
Full capacity ..... 7 254 000 000 gal.
Maximum depth ..... 65 ft.
Average depth ..... 19 ft.
Total length of shore including islands ..... 39 miles
Soil removed from reservoir ..... 4 593 000 cu. yd.
Roads discontinued ..... 8.42 miles
New roads built ..... 2.20 miles
Roads rebuilt ..... 5.24 miles
Area of watershed ..... 75.2 sq. miles

(See page 94 in 1898 report for quantities and cost.)

Wachusett Aqueduct.

First used in 1898.

Capacity ..... 362.6 m.g.d.

Total length ..... 12 miles

Rock Tunnel (12 ft. 2 in. wide, 10 ft. 10 in. high, lined) ..... 2 miles

Concrete and brick masonry covered with earth (11 ft. 6 in. wide, 10 ft. 6 in. high) includes Assabet River Bridge* ..... 7 miles

Open channel (20 ft. wide at bottom, sides 3:1) ..... 3 miles

Fall of 1:5000 in Tunnel from Station 1 + 41.85 to Station 100 + 00.

Fall of 1:2500 from Station 100 + 00 to Station 473 + 00.

Invert at Wachusett Dam ..... El. 284.52

Invert at Terminal Chamber ..... El. 267.63

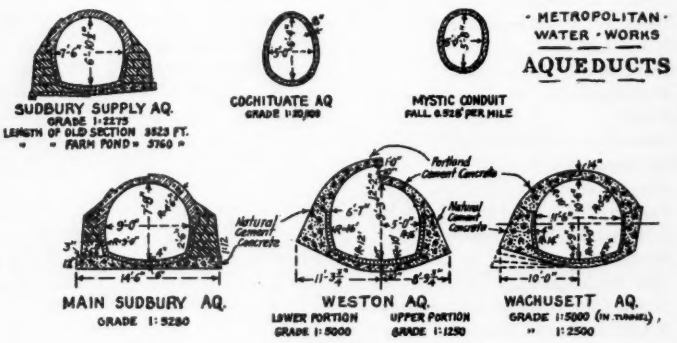

Fig. 26.

	Station.	Elevation of Invert.
Entrance of tunnel, Wachusett Dam	1 + 10.6	284.52
East end of tunnel	105 + 00	282.35
Gaging manhole	134 + 00	281.19
Assabet River Bridge	346 + 37.64	272.69
Terminal chamber at open channel	473 + 42.71	267.63
Intermediate Dam, open channel	520 + 00	264.60
Elevation of flashboards		272.6
Elevation of masonry crest		271.6
Lower Dam, open channel	632 + 25 ±	255.00
Elevation of flashboards		265.00
Elevation of masonry crest		264.00
Sudbury Reservoir at Lower Dam:		
Elevation of flashboards		260.00
Elevation of masonry crest		259.00
Elevation of bottom of reservoir		250.00

*Assabet River Bridge: Length (7 arches, 29 ft. 6 in. span) 359 ft. From surface of River to invert of Aqueduct, about 19 ft.

SUDBURY AQUEDUCT.

Built by city of Boston and taken by M. W. W. in 1898.

First used in February, 1878.

Capacity 109 m.g.d.

Framingham Dam No. 1 to Farm Pond Gate House, fall of 3.55 or 2.323 ft. a mile, size 7 ft. 6 in. wide at springing line × 6 ft. 10⅝ in. high for 1.5 miles.

Farm Pond Gate House to Chestnut Hill Reservoir, fall of 17.30 or 1 ft. a mile, size 9 ft. 0 in. wide at springing line × 7 ft. 8 in. high for 15.9 miles.

Total length ..... 91 850.1 ft. 17.38 miles

	Station.	Miles.	Elevation of Invert.
Framingham Dam No. 1	0		154.822
Farm Pond inlet chamber	41 + 67		152.990
Farm Pond Gate House (end of 7 ft. 6 in. × 6 ft. 10⅝ in. aqueduct)	79 + 18		151.269
Farm Pond Gate House (beginning 9 ft. 0 in. × 7 ft. 8 in. aqueduct)	1 + 00		151.352
Current-Meter Rating Station	59 + 00	1.1	150.216
Course Brook waste weir, Sherborn	112 + 00	2.1	149.212
Rockland St. Tunnel (beginning)	201 + 84	3.8	147.510
Bacon's waste-weir	271 + 40	5.2	146.193
Badger Hill Tunnel (beginning)	293 + 41	5.6	145.776
Waban Bridge (beginning)	381 + 97	7.3	144.079
Fuller's Brook waste-weir, Wellesley	459 + 00	8.7	142.620
West Siphon Chamber (beginning Rosemary Siphon)	540 + 00	10.2	141.086
East Siphon Chamber (end of Rosemary Siphon)	558 + 00	10.6	139.356
Charles River Bridge (beginning)	631 + 70	12.0	137.960
Sand Tunnel (beginning)	730 + 75	13.9	136.084
Clark's waste weir	738 + 15	14.0	135.944
Beacon Street Tunnel (beginning)	773 + 85	14.7	135.268
Terminal Chamber (end of aqueduct)	838 + 12	15.9	134.051

Waban Bridge, length (9 arches 44.67 ft. span) 536 ft., width 18 ft., height above bed of brook, 48 ft.

Charles River Bridge (Echo Bridge), length (7 arches) 475 ft., width 18 ft., height above bed of river, 79 ft., span of centre arch (radius 69.0 ft.), 129 ft.

Rosemary Siphon which crosses Rosemary Brook consists of 2 lines of 48 in., 1 line of 61 in. C. I. pipe, each line 1 800 ft. long.

There are 4 waste-weirs and 44 culverts on the aqueduct.

Cochituate Aqueduct.

Built by City of Boston, taken by M. W. W. in 1898.
First used in October, 1848.
Capacity 20 m.g.d.
Fall of 4.26 ft., or 3⅙ in. a mile, size 5 ft. wide at springing line × 6 ft. 4 in. high.
Length 13.7 miles.

	Station.	Miles.	Elevation of Invert.
Effluent Gate House			131.030
Dedman's Brook waste-weir	157 + 47	3.0	
Current Meter Rating Station, Wellesley	286 + 58	5.4	
Grantville waste-weir	350 + 59	6.6	
Beginning of 40 in. Siphon Pipe	411 + 18	7.8	
West Pipe Chamber (beginning Charles River Siphon)	412 + 64	7.8	129.040
East Pipe Chamber (end Charles River Siphon)	423 + 33	8.0	128.520
End of 40 in. siphon pipe	425 + 12	8.1	
Newton Center waste-weir	573 + 74	10.9	
Ward Hill Tunnel (beginning)	619 + 34	11.7	
Ventilator	646 + 80	12.3	
Connection to Influent Gate House, Chestnut Hill Reservoir	684 + 51	13.0	
Intermediate Gate House (west side)	698 + 35.3	13.2	127.17
Pipe connection from Sudbury Aqueduct Terminal Chamber	700 + 72.5	13.3	
Connecting Chamber, High Service Pump Station.	720 + 66	13.7	

Remarks: Charles River Siphon consists of 4 lines of C. I. Pipe, 2-30-in. and 1-36-in., 1 068.46 ft. long and 1-40-in. line 1 393.89 ft. long.

Weston Aqueduct.

Built by Metropolitan Water Works.

First used in December 1903.

Capacity, 300 m.g.d.

Head House to siphon under N. H. R.R. at Nobscot: concrete aqueduct, fall of 14.48 ft. or 4.224 ft. a mile, size 10 ft. 0 in. wide at springing line × 9.2 ft. high for 3.43 miles. s = 1: 1250.

Siphon under N. H. R.R. at Nobscot: 130.2 ft. long, fall 2.92 ft.

Siphon at Nobscot to Channel Chamber: fall of 10.57 ft. or 1.056 ft. a mile, size 13.17 ft. wide at spring line, × 12.17 ft. high for 7.79 miles; s = 1:5 000.

Total length 13.42 miles.

	Station.	Miles.	Elevation of Invert.
Head House (end of pipes, entrance to Head House)	5 + 06		215.89
Tunnel No. 1 (beginning)	7 + 00		
Gaging Chamber No. 1	17 + 00	0.2	214.97
Tunnel No. 2 (beginning)	42 + 30	0.7	212.96
Tunnel No. 3 (beginning)	113 + 20	2.0	
Nobscot Siphon (beginning)	186 + 50	3.4	201.41
Nobscot Siphon (end)	187 + 80	3.5	198.49
Gaging Chamber No. 2	214 + 21	5.0	197.97
Siphon Chamber No. 1 (beginning of Sudbury River Siphon)	284 + 68	5.3	196.56
Siphon Chamber No. 2 (end of Sudbury River Siphon)	320 + 74	6.0	194.15
Siphon Chamber No. 3 (beginning of Happy Hollow Siphon)	353 + 53	6.6	193.46
Siphon Chamber No. 4 (end of Happy Hollow Siphon)	364 + 77	6.8	192.57
Tunnel No. 4 (beginning)	529 + 61	10.0	
End of Aqueduct (Channel Chamber)	597 + 38	11.3	187.92
Ash St. Bridge, East End	611 + 47 ±	11.6	187.50
Screen Chamber, West End	649 + 88	12.3	187.90
Terminal Chamber (end of aqueduct)	706 + 99	13.42	187.90
Entrance to Distributing Pipes	706 + 99.4		187.50

*Sudbury River Siphon steel pipe is 7.5 ft. in diameter and 3 605.55 ft. long.

*Happy Hollow Siphon steel pipe is 7.5 ft. in diameter and 1 124.02 ft. long.

Sudbury River Siphon crosses the river by arching the pipe to carry itself and the weight of the water; rise of pipe 5.5 ft.; length between abutments 80.0 ft.

Open Channel is 11⅓ miles from Sudbury Dam and is 1 400 ft. long, 12 ft. deep and 10 ft. wide at bottom, slope of sides 3:1.

*Provision for three lines of pipe, only one line laid.

SUDBURY POWER PLANT.

Regular service, 2 units installed, September 14, 1916.
Regular service, 3 units installed, November 27, 1916.
Hydraulic Machinery:
2–24 in. 325 h.p. vertical, shaft hydraulic turbines.
1–30 in, 1 000 h.p. vertical shaft hydraulic turbines.
Furnished by S. Morgan Smith Co., York, Pa.
30 in. on test at Holyoke, developed 87.4% efficiency.
24 in. on test at Holyoke, developed 85.3% efficiency.
Cost — $13 410.95, wiring, etc. $1 291.71.
Electrical Equipment, furnished by Westinghouse Electric & Mfg. Co., of Pittsburg, Pa.
1–800 k.v.a. alternating current generators.
2–250 k.v.a. alternating current generators.
At 360 r.p.m. to deliver 3-phase, 60-cycle, A.C.
At 2 400 volts, stepped up to 13 200 volts, and
2–750 k.v.a. transformers.
2–125 v.d.c. generators for exciters.
One exciter direct connected with electric motor, the other direct connected to top of shaft of 900 k.v.a. generator.
Cost of generators, $20 859.76.
Cost of lightning arrester chamber, $1 175.17.
3 Horizontal type hydraulic governors from Lombard Governor Co., $2 516.06.
1 of 6 000 ft. lb. capacity.
2 of 2 500 ft. lb. capacity.
Hand-operated travelling crane, 12 000 lb., cost $880.
2 Surge tanks for Weston Aqueduct 15 ft. × 7½ ft. × 7 ft. high.
1 Surge tank for Reservoir No. 3, 10 ft. × 6 ft. × 13 ft. high.
Cost of wheel pits and tanks, $46 906.14.
Overhead transmission line, 4 170 ft. long.
Underground cable 1 500 ft. paper-insulated lead-sheathed, 3 conductors, tested to 30 000 volts.
Total cost of whole equipment, $102 103.

DISTRIBUTING RESERVOIRS.

	High Water.	Available Capacity. Mil. Gals.	Area. Acres.	Earth Embankment.	Invert of Effluent Pipe.	Reservoir First Filled.	Maximum Depth. Feet.	Owned by.	Remarks.
Spot Pond Reservoir	163.00	1 791.7	308.7	166.00	137.80 N. 137.00 S.	1901	47	Met. W. W.	.....
Chestnut Hill { Lawrence Basin		168.6	37.7				17		
Chestnut Hill { Bradlee Basin	134.00	522.8	85.0	138.00	109.80	1868	32	Met. W. W.	1870
Weston Reservoir	200.00	200.0	60.0	205.00	187.90	....	27	Met. W. W.	....
Fells Reservoir	271.00	41.353	8.5	275.00	249.75	1899	21	Met. W. W.	....
Mystic Reservoir 560 x 350 ft.; 1½:1 slope	157.00	26.24	4.5	160.00	135.50	1863	22	Met. W. W.	....
Waban Hill Reservoir	264.50	13.50	2.8	270.00	246.00	1877	16.7	Met. W. W.	Bought Oct. 12, 1900
Forbes Hill Reservoir 150 x 350 ft.; 1¾:1 slope	192.00	5.116	1.226	195.00	174.50	1901	17	Met. W. W.	....
Bear Hill Reservoir	300.00	2.452	0.767	303.50	285.00	1902	15	Met. W. W.	....
Fisher Hill Reservoir 500 x 295 ft.	251.00	15.50	3.14	255.00	230.00	1887	18	Met. W. W.	1877 Bought 1913
Chelsea Reservoir 178 x 98 x 15 ft.	196.60	1.00	0.40	199.40	....	....	15	Chelsea	....
Revere Reservoir 150 x 156 x 18.6 ft.; 2:1 slope	194.50	1.818	0.54	195.10	.....	....	18.6	Revere	....

STORAGE RESERVOIRS AND DAMS.

	Drainage Area Above Outlet.	Area of Water Surface.	Capacity at High Water.	Maximum Depth of Water.	Length of Dam.	Length of Overfall.	Total Height of Dam.	Elevation Above Boston City Base.				Reservoir First Filled.
								Earth Embankment.	High Water.	Top of Flashboards.	Stone Crest.	
	Sq. Mi.	Acres.	Mil. Gals.	Feet.	Feet.	Feet.	Feet.	Feet.	Feet.	Feet.	Feet.	
Wachusett	108.84	4 135	64 968.0	129	1 423	452	207	N. Dike 412 S. Dike 410-414	395.00	398.00	395.00	1908
Sudbury	22.28	1 292	7 253.5	65	1 940	300	..	266.00	260.00	260.16 259.67	259.00	1898
Framingham No. 1.	74.66	154	311.0	16	785.8	168.25	22	176.80	169.32	169.27 168.42	167.60	1879
Framingham No. 2.	45.14	134	562.6	20	1 340	184.38	26	183.83	177.87	177.12 176.49	175.87	1879
Framingham No. 3.	27.68	250	1 199.7	24	3 092	100.25	29	191.32	186.74	186.50	185.24	1878
Ashland	6.43	169	1 416.4	48	1 857	30	58	231.00	225.21	225.21 224.90	224.23	1886
Hopkinton	5.86	193	1 520.9	53	1 540	30	59	311.00	305.00	305.00 304.67	304.00	1895
Whitehall	4.35	601	1 256.9	18	..	..	13	339.90	337.91	Coffer Dam 339.91	Bottom of Gates 327.78	Natural
Lake Cochituate	17.58	730	2 097.1	70	125	63	..	148.36	144.36	144.40	142.05	Natural 1848
Farm Pond	0.54	165	167.5	12	1 115	..	12	164.00	159.25	.....	.....	....

STANDPIPES IN METROPOLITAN DISTRICT.

Location.	Dimensions. Diameter.	Dimensions. Height.	Capacity Brim Full Gallons.	Gallons per Foot.	Elevation at Top.	Elevation at Bottom.	Owned by.	Date Erected.
Arlington	75 ft.	61 ft. 3 in.	2 000 000	33 045	443.25	382	Met. W. W.	1921–22
Bellevue	100 ft.	44 ft. 3 in.	2 500 000	58 700	376.25	332.00	Met. W. W.	1914–15
Forbes Hill	30 ft.	64.33 ft.	330 000	5 287	251.00	188.5	Met. W. W.	1900
Lexington	30 ft.	105 ft.	550 000	5 287	443.43	.....	Lexington	1912
Nahant	30 ft.	95 ft.	502 300	5 287	170.0	75.0	Nahant	1902
Quincy	35 ft.	60 ft.	425 000	7 200	249.0	189.0	Quincy	.....
Swampscott	40 ft.	65 ft.	611 000	9 400	200.0	.....	Swampscott	.....
Wakefield	40 ft.	62.3 ft.	585 600	9 400	296.0	.....	Wakefield	1883
Winthrop	40 ft.	100 ft.	940 000	9 400	204.0	.....	Winthrop	1910
Orient Heights	24 ft.	40 ft.	133 364	3 384	204.4	.....	Boston	1889

Pumping Engines.

Station.	Eng. No.	Diam. of Steam Cylinders.	Diam. Plunger	Stroke.	R.P.M.	Cu. Ft. per Rev.	M.G. 24 H.	Speed Ft.Min	H.P.	Head Ft.	Suct. In.	Dis. In.	Erected	Cost.	Designer.	Builder.
Chestnut Hill No. 1	3	13.7–24⅜–39	17.5	48	50	39.91	20	400	550	137.5	48	36	1895	140 000	E. D. Leavitt	Quintard Iron Works
Chestnut Hill No. 1	4	30–56–87	42	66	17.5	158.75	30	192.5	800	140	48	48	1898	139 562	E. P. Allis Co.	
Chestnut Hill No. 1	16	23–54	23.75	36	38.9	35.25	15	203.4	448	133	36	30	1922	75 900	Worthington St. Pump Corp.	
Chestnut Hill No. 1	19	a.			525	8.845	50	....	1 390	135	42	36	1932	41 816	Warren St. Pump Co. and	
Chestnut Hill No. 1	20	b.			900	1.547	15	....	623	200	24	24	1932	22 385	Westinghouse Elec. & Mfg. Co.	
Chestnut Hill No. 2	5–6–7	17–31¼–48	37	60	29	112	35	290	960	45	36	36	1899	45,500 ea.	Holly Manufacturing Co.	
Chestnut Hill No. 2	12	32–60–90	43.5	60	24	154.81	40	240	1 036	132	36	36	1911	96 769	Holly Manufacturing Co.	
Arlington	10	14–28	9.5	18	63	2.78	1.89	186	76	290	12	12	1907	6 370	Allis Chalmers Co.	
Arlington	15	c.			1675	0.165	3	....	171	320	12	10	1918	9 700	F. A. Mazzur	Allis Moore Co.
Arlington	18	16–32	10.375	30	51	5.45	3	255	178	305	16	16	1928	26 119	Murray Iron Works	
Spot Pond	8	21–42	14⅞–21	48	51	19.24	10.57	204	230	125	36	30	1900	.....	Taken from City of Boston	
Spot Pond	9	22–41½–62	30.5	60	24.4	76.17	20	244	463	125	30.5	36	1900	62 500	Holly Manufacturing Co.	
Spot Pond	17	24–54	26.5	36	41.6	45.30	20	249.6	479	125	42	36	1925	6 900	Worthington St. Pump Corp.	
Hyde Park	13–14	13–27	11.5	24	50	5.58	3	200	85	140	20	16	1913	8 190 ea.	Laidlaw Dunn Gordon Co.	
Pegan	1–2	5–10	d		300	1.238	4	....	25	11	8 10	8 10	1908	750 ea.	Fore River Ship and Engine Co.	

a.–b. 19–20 Warren Centrifugal Pumps driven by Westinghouse steam turbines.
c. 15 Two 8–in. Allis Chalmers Centrifugal Pumps in series driven by Moore steam turbine.
d. 1–2 8 and 10 in. Centrifugal Pumps driven by Marine Engines.

BOILERS.

Station.	No.	Type.	Shell.				Tubes.			Heating Surface		Steam Noz'ls	Grate.	Builder.	Erected.
			Diam. In.	Length Ft. In.	*Pressure.	Thickness. In.	No.	Diam. In.	Length. Ft. In.	Water. Sq. Ft.	Total. Sq. Ft.	Diam. In.	Area. Sq. Ft.		
Chestnut Hill No. 1	20–21	Ver.	98	24 0¾	189	15/16	400	2	15 0	2 238	3 028	5	42.12	Dillon Steam Boiler Works	1921
Chestnut Hill No. 1	22	Ver.	98	24 0¾	189	15/16	400	2	15 0	2 238	3 028	5	42.12	Coatesville Boiler Works	1922
Chestnut Hill No. 1	26–27	Ver.	98	24 0¾	189	15/16	392	2	15 0	2 207	2 990	5	42.12	International Eng. Co.	1931
Chestnut Hill No. 1	28	Ver.	98	24 0¾	189	15/16	392	2	15 0	2 207	2 990	5	42.12	Dillon Steam Boiler Works	1932
Chestnut Hill No. 2	15–16	Ver.	109⅞	24 1½	166	⅞	484	2	15 0	2 704	3 638	5	54.54	Robb Mumford Boiler Co.	1911
Chestnut Hill No. 2	29–30	Ver.	98	24 1½	166	15/16	392	2	15 0	2 207	2 990	5	42.12	Dillon Steam Boiler Works	1932
Chestnut Hill No. 2	31	Ver.	98	24 1½	166	15/16	392	2	15 0	2 207	2 990	5	42.12	Dillon Steam Boiler Works	1932
Arlington	13–14	Hor.	54	17 5½	160	7/16	60	3	17 0	873	873	4	22	Hodge Boiler Works	1907
Arlington	19	Hor.	54	17 5½	160	15/32	60	3	17 0	873	873	4	22	New England Iron Works	1918
Spot Pond	23	Ver.	98	24 0¾	189	15/16	400	2	15 0	2 238	3 028	5	42.12	Coatesville Boiler Works	1923
Spot Pond	24	Ver.	98	24 0¾	189	15/16	392	2	15 0	2 207	2 990	5	42.12	Dillon Steam Boiler Works	1930
Spot Pond	25	Ver.	98	24 0¾	189	15/16	392	2	15 0	2 207	2 990	5	42.12	International Eng. Co.	1931
Hyde Park	17–18	Hor.	54	14 5⅞	150	7/16	62	3	14 0	748	748	4	20	Robb Engineering Co.	1912
Pegan	1–2	Ver.	54	11 0	150	7/16	148	2	7 5	391	573	2.5	12.56	Dillon Steam Boiler Works	1923

*Pressure, pound per square inch.

DUTIES OF PUMPING ENGINES.

Station.	Engine No.		Date of Test.
Chestnut Hill No. 1	3	150 million ft. lb. per 100 lb. of coal	1895
Chestnut Hill No. 1	4	176 million ft. lb. per 1 000 lb. of steam	1900
Chestnut Hill No. 1	16	148 million ft. lb. per 1 000 lb. of steam	1922
Chestnut Hill No. 1	19	160 million ft. lb. per 1 000 lb. of steam	1933
Chestnut Hill No. 1	20	144 million ft. lb. per 1 000 lb. of steam	1933
Chestnut Hill No. 2	5–6–7	159 million ft. lb. per 1 000 lb. of steam	1901
Chestnut Hill No. 2	12	175 million ft. lb. per 1 000 lb. of steam	1912
Arlington.....	10	116 million ft. lb. per 1 000 lb. of steam	1909
Arlington.....	15	90 million ft. lb. per 1 000 lb. of steam	1918
Arlington.....	18	142 million ft. lb. per 1 000 lb. of steam	1928
Spot Pond.....	8	127 million ft. lb. per 100 lb. of coal as No. 4 at Mystic	1895
Spot Pond.....	9	172 million ft. lb. per 1 000 lb. of steam	1901
Spot Pond.....	17	142 million ft. lb. per 1 000 lb. of steam	1925
Hyde Park.....	13	120 million ft. lb. per 1 000 lb. of steam	1913
Hyde Park.....	14	121 million ft. lb. per 1 000 lb. of steam	1913

PROCEEDINGS.

MARCH MEETING.

HOTEL KENMORE, BOSTON, MASS.

Wednesday, March 21, 1934.

President E. Sherman Chase in the Chair.

Secretary GIFFORD. At the Executive Committee meeting this morning the following were elected members: Fred O. Walker, Superintendent, Rumford, Me.; Ralph Adams, Chairman, Board of Public Works, Arlington, Mass.; George B. Mebus, Designing Engineer, Glenside, Pa.

The PRESIDENT. I regret to announce the death on Sunday of Mr. Charles E. Pratt of the Chapman Valve Company. Mr. Pratt was known to most of us as an active participant in these meetings for a great many years. Will the members please rise for a moment in respect of the memory of Mr. Pratt.

(All the members rise and stand in silence.)

The Report of the Committee on Hydrant Markings was read by Mr. Harry U. Fuller, Chairman. (See p. 252.)

On motion, duly seconded, it was VOTED that the report of Committee on Hydrant Markings be accepted and placed on file.

A series of motion pictures illustrating Civil Works Administration water-works construction in Leominster, Mass., was shown and explained by W. Guy Classon, Superintendent and Registrar, Leominster Water Works.

A discussion on Cold Weather Troubles was opened by Percy A. Shaw of Manchester, New Hampshire, and participated in by Messrs. George E. Hawkins, Harry U. Fuller, David A. Heffernan, W. Guy Classon, Henry T. Gidley, Frank J. Gifford, George McKay, Hervey A. Hanscom, Edmund C. Sanderson, William F. Sullivan, Samuel A. Agnew, Henry E. Halpin, Charles W. Mowry, Roger W. Esty, Albert A. Ross, and Harold L. Brigham.

(*Adjourned*)

REPORT OF COMMITTEE ON HYDRANT MARKINGS.

To the New England Water Works Association:

Your committee on the Standard Markings of Fire Hydrants submits the following report:

This subject is of importance to all water utilities and fire departments. If every hydrant in a water works is tested to learn the amount of water it will deliver at a given residual pressure and the hydrant is marked to show the result, the fire department will be able to work to better advantage and the water-works officers will have valuable records. If the system of marking is uniform, fire departments from other towns will be able in time of emergency to locate at the proper hydrants. It is desirable that the capacity of all hydrants be determined at the same residual pressure and that the full capacity when supplying pumpers be known. For this reason a residual pressure of 10 lb. per sq. in. has been adopted for the standard.

Markings. Markings should be determined by flow test, and should be divided into three classes, A, B, and C.

Class A. Hydrants which have sufficient capacity to supply 1 000 or more gallons per minute.

Class B. Hydrants which have sufficient capacity to supply not less than 500 gallons per minute.

Class C. Hydrants with insufficient capacity to supply 500 gallons per minute.

Post Hydrant Markings. Barrels of all hydrants should be painted chrome yellow for visibility.

Nozzle and Cap Markings:

Class A. Tops and nozzle caps should be painted green.

Class B. Tops and nozzle caps should be painted yellow.

Class C. Tops and nozzle caps should be painted red.

The colorings conform to standard traffic signals with green the most desirable, and red the least desirable.

Flush Hydrant Colorings. All location markers for flush hydrants should carry upon their background, the colors designating the class in which that particular hydrant falls.

All flush hydrant covers should carry upon the under side, preferably within the circle, the proper color as a background upon which any necessary data could be stencilled in black.

Respectfully submitted,

(Signed) HARRY U. FULLER,
GEORGE H. FINNERAN,
FRANK E. WATERMAN.

Remarks by Mr. Harry U. Fuller.

Some question has been raised as to the amount of information that it is proper to attempt to convey by marking hydrants. Of course it is desirable to convey all the information possible, but it is not desirable to convey misinformation. What should be done is controlled by the degree of accuracy of the information conveyed. If the hydrants are marked as suggested in the report the information is not perfectly accurate because the number of nearby hydrants being used affects the results.

It has been suggested that hydrants be tested in groups instead of singly, but if that were done there is no standard of how many to use or which ones to use. One of the advantages of testing the hydrants singly is that the tests can be repeated at intervals and the effect of improvements to the system can be learned as well as reduced capacity due to the age of the pipes.

It has been suggested that the size of the main be painted on the hydrant, but if the hydrant were on a 6-in. main the supply might be one fire stream or six, depending on the length of 6-in. supply. That information would be of no help.

Another suggestion has been made that more colors or more figures be used to represent smaller differences in quantity. I should call that an unreasonable attempt at refinement and not justified. It seems to me that the suggested report is as far as we should go at this time. In those places where I have known of the suggested marking being applied it has been well received by the fire departments and they consider the idea good.

When the capacity of the hydrant is greater than 2 000 gallons per minute, a number should be painted on the barrel of the hydrant signifying the number of thousand gallons per minute exceeded.